EUROPEAN NO•DIG 2021
UNITED KINGDOM

1st International NODIG Conference

"Rehabilitation Design for Pressure and Gravity Pipes"
15 September 2021 <>Peterborough Arena, UK

1st International NODIG Conference

INTRODUCTION

The idea for a European No-Dig conference grew from discussions between UKSTT, GSTT, FSTT and IATT at the ISTT International Conference in Florence in 2019. As a result of these discussions UKSTT agreed to host the first Conference, planned for 2020 alongside the biennial No-Dig Live event. Covid intervened so both the No-Dig Live and the Conference were postponed to September 2021. Travel was still difficult at that time but the authors all worked very hard to make the Conference a success, some online and some in person.

A Conference is only as good as its content. With this in mind we established a Technical Committee to ensure a high standard. I was privileged to be the Chairman of this Committee and am grateful for the support of my fellow members: Prof. Stefano Mambretti (IATT), Dr. Olivier Thépot (FSTT), Julian Britton (UKSTT), Dr. Dec Downey (ISTT) and Jens Hölterhoff (GSTT). All of them gave their time generously to review submissions during a difficult period for everybody.

The content of a Conference depends on the authors. Our objective was to focus on design of liners in both gravity and pressure applications. The keynote authors, Dr. Thépot and Dr. Gumbel, established a very high technical level which set the scene very well and the other authors' papers each focused on a specific aspect, often practical, to support and emphasise certain key points. On behalf of the Technical Committee and of UKSTT I thank them all for their magnificent efforts in preparing the papers and presentations.

The content of these Proceedings will remain valid and relevant for many years and I consider this 1st European Conference to be a milestone in the development and growth of trenchless technologies in Europe. Already in May 2023 a successful 2nd European Conference has been held in Milan, organised by IATT. I look forward to future Conferences in the series to build upon this strong foundation and to disseminate technical knowledge and understanding throughout and beyond the trenchless community.

Tom Sangster

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Session 1

GRAVITY SEWER REHABILITATION

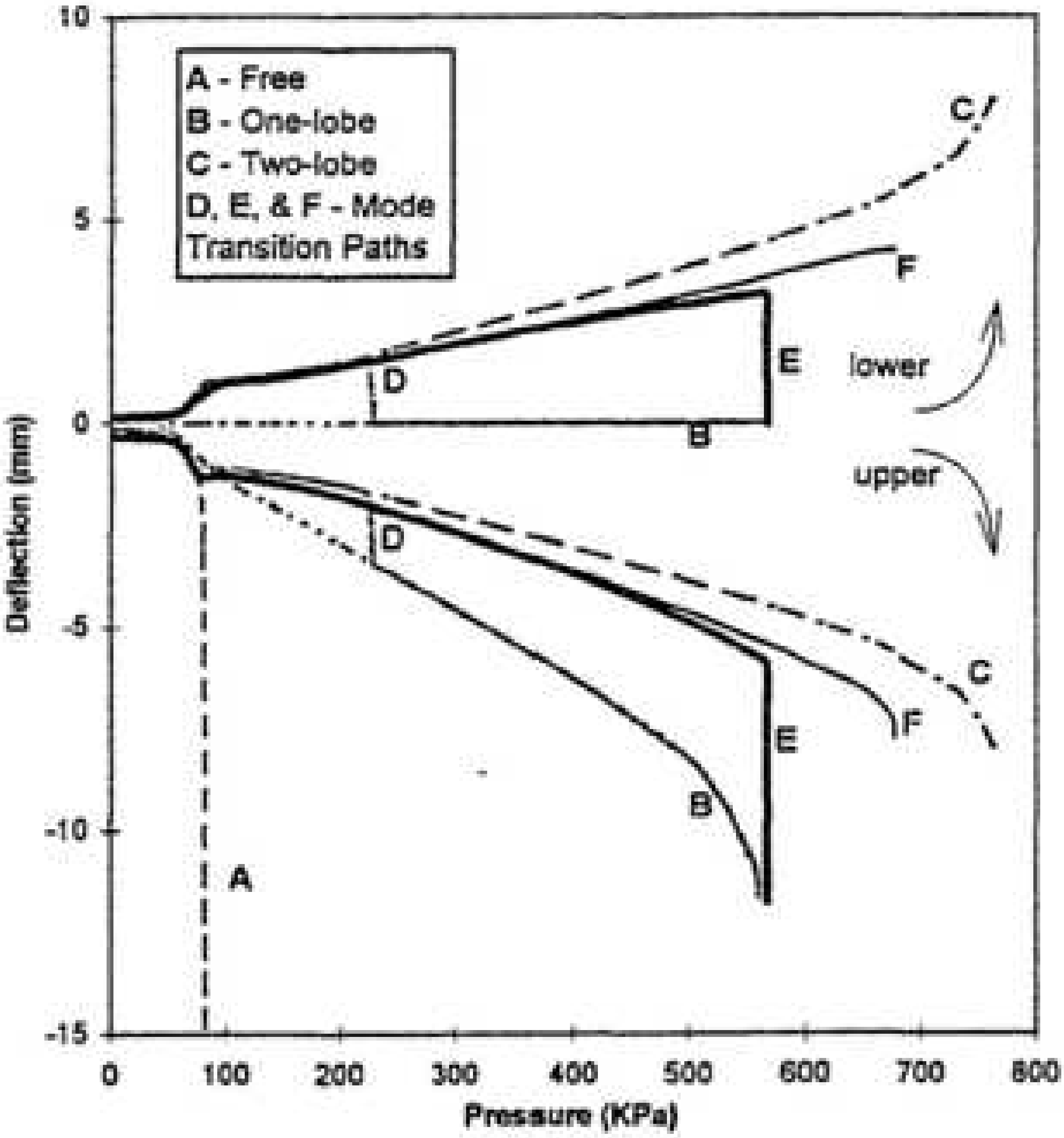

Figure 3. Possible transition paths from Zhao 1999.

1st European No-Dig Conference 2021

Weds 16 June, Peterborough, UK

Session 1 – GRAVITY SEWER REHABILITATION

Keynote lecture – Gravity sewer Liner Design

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Abstract We present a summary of the tests which have been carried out on liners (close fit and slip liners) since the beginning of the 1990s, first those carried out by Guice et al. then David Hall at the Trenchless Technology Center of Louisiana Technical University (USA), those of J. Boot at the University of Bradford (England), those carried out by Ian Moore and his teams at Queen's University (Canada), and also those carried out by Prof. Bernard Falter at the University of Munster (Germany). All these tests have made it possible to highlight the real failure modes and the strength limits of liners placed in intact or cracked rigid circular pipes, ovalized pipes, non-circular pipes, flexible corrugated steel pipes, under service and ultimate loads, subject to external hydrostatic pressure, earth loads and vehicule loads.

The resistance of close-fit liners to external hydrostatic loading is now very well understood and analytical models derived from the works of Glock for circular liners and Thepot for non-circular liners are now integrated in the most advanced national method. The data from the tests showed that the predictions of the Glock's model which natively considers the interaction between the lining and the host pipe, are more accurate than those of an unsupported ring corrected by an enhancement factor used in ASTM F1216. In addition, the Glock's model allows to take into account imperfections (gap, elliptical ovality, 4-crack ovality, flat section, intrusion) and can be extended to non-circular liner.

Misconceptions concerning how loads apply on flexible close-fit liners has been corrected in particular thanks to the experimental research of I.D. Moore and his teams who have shown that loads transmitted by the soil-damaged host pipe to close fit liners are mainly imposed displacements which lead to local bending without any developing of hoop force that exclude general earth buckling. Law and Moore introduced the concept of "deferred ovalisation" which capture the impact of vertical diameter decrease after lining, and developed a remarkably simple analytical model for liner encased in cracked pipe which has been integrated in the French national method and generalized to non-circular cracked pipe in a future ASCE manual of practice.

EXPERIMENTS ON RESISTANCE OF PIPE LINERS TO EXTERNAL FLUID PRESSURE

Tests of Aggawarl and Cooper (1984)

Experiments on liners has primary focused on the resistance to buckling failure under external fluid pressure.

The instantaneous buckling of CIPP liners was first tested by Aggawarl and Cooper in 1984 on 49 Insituform CIPP liners. Liners were cast within circular steel tube and subjected to external water pressure.

They observed that the buckling pressures were much higher than those of a freestanding tube.

To account for this observation, they introduced an enhancement factor K equals to the ratio of the experimental buckling pressure to the theoretical buckling pressure of a free standing tube given by the Timoshenko formula.

$$K = \frac{P_{cr-exp}}{P_{cr0}}$$

with

$$P_{cr0} = \frac{2E}{(1-\nu^2)} \cdot \frac{1}{(DR-1)^3}$$

Where $P_{cr\text{-}exp}$ = experimental buckling pressure; K = enhancement factor; P_{cr0} = theoretical buckling pressure of a free standing tube (Timoshenko formula).

From 49 specimens with Diameter ratios (D/t) ranging from 30 to 90, they found that the enhancement factors varied from 6.5 to 25.8.

Aggarwal and Cooper concluded that since 46 of the 49 tests gave a value of K greater than 7.0, that 7.0 should be the 95% lower confidence limit of K to use for design purpose.

This factor with the Timoshenko formula was adopted for CIPP design in the WRc manual and in the appendix X1 (under nonmandatory information) of the ASTM F1216 standard (ASTM 1993).

Note that it was already fairly obvious despite the dispersion of the results that K increased with the ratio D/t.

If the K enhancement factor empirically solved the problem of the interaction between the liner and the host pipe, it remained another even greater difficulty linked to the creep deformation shown by thermosetting polymers materials used for liners.

Creep deformation increases with time (and is added to elastic deformation) and decreases buckling resistance. The buckling resistance then becomes dependent on time and therefore on the duration of the application of the external pressure. The duration of 50 years was considered to be a standard for the expected service life.

A simple way to take into account creeping is to use an apparent long-term flexural modulus (or time corrected modulus) determined on beam samples by a long-term tensile test or flexural test under constant load were the "creep modulus" is measured over 10,000 h with extrapolation to 50 years. However this method is not entirely satisfactory because it supposed a linear viscoelastic and isotropic material and a constant or slowly varying stress, but these conditions are never verified for a liner. Indeed thermosetting polymers used in CIPP liners exhibit nonlinear and anisotropic behavior: the creep rate for compression is often much lower than that associated with tension, and despite that the external hydrostatic pressure is constant, the bending stress greatly increases with the deflections of the liner.

Despite all its limitations the time corrected modulus was adopted in the WRc and ASTM F1216 methods because it was relatively easy to measure with existing test. It had even become a common industry practice to set the long term modulus of elasticity at 50% of the short-term modulus of elasticity for a 50 years design life and for all CIPP materials.

Research conducted at Trenchless Technology Center, Louisiana Tech University

A major joint industry/university/government research program was launched in the early 1990s to evaluate the long-term structural performance of liners. This research program took place at Louisiana Tech University under the direction of Leslie K. Guice and W. Thomas Straughan (Guice 1994).

K. Guice and W.T. Straughan believed that using a time corrected modulus was fundamentally incorrect and that the correct way was to measure on liners installed in steel casings the time required for buckling (failure) and the corresponding external water pressure, such as the test method for Plastic pipe failure time under constant internal pressure.

A test program was designed to simulate the sustained external hydrostatic loading that would be experienced by liners constrained in partially deteriorated gravity pipes. Each test specimen remained under constant pressure for up to 10,000 hours or until failure.

The test method used a similar approach to that of ASTM D2837 for plastic pipes under long-term internal pressure.

Six different products, including five CIPP and one FFP (Fold and Form Pipe) were evaluated in about 200 tests conducted on 12 in diameter pipes with dimension ratios ranging from 30 to 65. Two products (Insituform Standard and Enhanced) had 40 samples each to produce statistically significant data.

Tensile and flexural tests were conducted to characterize the materials (modulus of elasticity and strength). Preliminary short-term tests were conducted to establish instantaneous buckling pressures for the encased liners. Using the instantaneous buckling pressure as an upper limit, at least four groups of different long term test pressures for each product were selected.

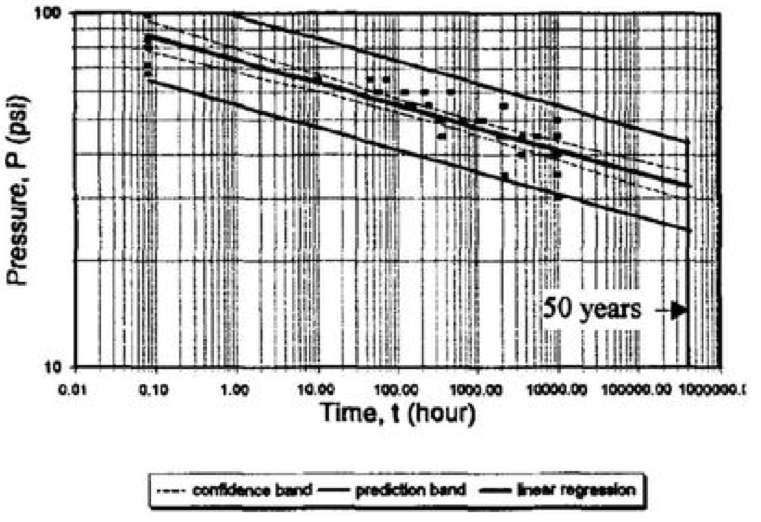

Figure 1. Long-term test and regression results from TTC Report 302, 1994.

Regression analyses was performed on external pressure versus time of failure using the following equation:

$\text{Log}(P) = a - b \log(t)$

The linear regression curves were extrapolated to 50 years to provide the pressure that could be sustained for the service life (Figure 1).

The extrapolated to 50 years test buckling pressure was compared to the predicted buckling pressure calculated with the modified Timoshenko equation (used in Appendix X1 of ASTM F1216) with a enhancement factor K of 7.0 and a time corrected modulus set to 50% of the initial modulus of elasticity.

The ratio of the 50 years test buckling pressure to the ASTM buckling pressure was greater than 1.0 for the four CIPP polyester products, varying from 1.16 to 1.46 (for the Insituform enhancement product) with an average value of 1.28. This means that the ASTM formula with a creep factor of 0.5 and an enhancement factor of 7 was slightly conservative (without consideration of safety factor) for polyester resin. The back calculated creep factor (calculated by equating the ASTM formula with the test result) varied from 0.58 = 1.16x0.5 to 0.73=1.46x0.5 with an average value of 0.64.

However, 2 products had a ratio lower than one: a CIPP with Vinyl ester resin (ratio = 0.76, back calculated creep factor = 0.38) and a PVC Fold and form pipe (ratio = 0.78, back calculated creep factor = 0.39).

Even if this was not the primary objective of Leslie K. Guice and W. Thomas Straughan, the ASTM equation with the enhancement factor of 7 and the creep factor of 0.5 was rather validated by the tests.

Notice that long-term flexural tests was not carried on in the research program since Guice and Straughan considered that the use of the time corrected modulus was fundamentally incorrect to predict creep buckling. So it was not possible to compare the creep reduction factor resulting from the long term bending test to that resulting from the hydrostatic buckling test.

This comparison was carried out but indirectly in a research program conducted by the Center for Advancement of Trenchless Technologies of the Waterloo University (Knight, 2005).

A.M. Riahi (2015), compared the creep factors measured at TTC by hydrostic tests on 2 CIPP materials (Insituform products) with those measured by bending tests according to ASTM 2990 on "similar" materials. He concluded that both methods yield similar results. **However, ASTM D2990 provides a much simpler and less expensive test procedure than the hydrostatic buckling test.**

In 1998, a new buckling test program was carried out at the TTC, Louisiana Tech University (TTC 1998), to investigate the short and long term buckling behavior of CIPP liners encased in circular (12" and 8" ID) and oval pipes. The main goal of the TTC research program was to improve the ability to predict the short-term and long-term behavior of liners by developing models capable of explicitly taking into account the various geometrical and mechanical parameters. And it was a complete success thanks to the modeling works of Qiang Zhao and David Hall of the Technical University of Louisiana.

Q. Zhao used the finite element method very extensively to analyze test results taking into account the main sources of nonlinearity: time-dependant material deformation (creep), yield deformation, variable contact surface and friction. A particular CIPP product was chosen, three geometric parameters was investigated: the dimension ratio of the liner (D/t), the gap between the liner and the host pipe, and the ovality (elliptical) of the host pipe.

The main results of the short-test are as follows:

Buckling mode:

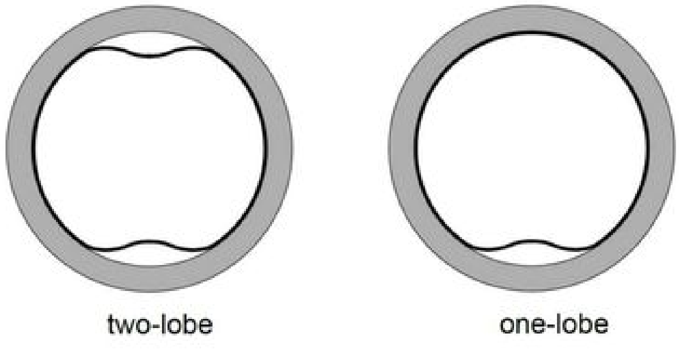

Figure 2. Typical buckling modes.

Experimental results indicate an initial two-lobe buckling mode which ultimately leads to single-lobe collapse suggesting that liner deflection could undergo a transition from two-lobe mode to one-lobe mode.

The finite element method shows that the critical pressure of the bi-lobe mode is approximately 30% higher than the critical pressure of the mono-lobe mode.

The finite element method also shows that the initial deflection mode is bi-lobe but that this mode is unstable and can branch off towards the mono-lobe mode. The transition occurs more or less quickly depending on the coefficient of friction (Figure 3).

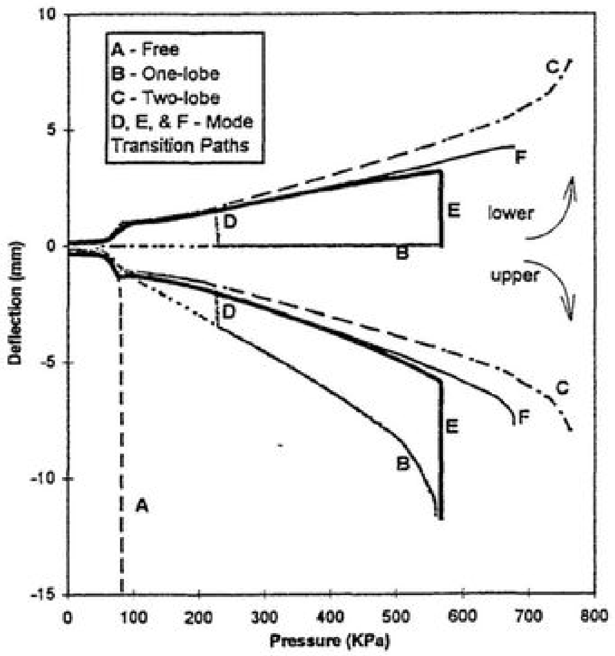

Figure 3. Possible transition paths from Zhao 1999.

When there is little or no friction between the liner and the host pipe, the transition from a two-lobe to a one-lobe deflection pattern will occur in an early stage of lobe development and the final buckling pressure will be close to that of the mono-lobe mode (paths E in figure 3). Conversely significant friction between the liner and the host pipe will prevent early transition and the final buckling pressure will be close to that of the two-lobe mode (paths F in figure 3).

Concerning the stress evolution, **the plastic yield strengths (associated with tensile and compressive stresses) are reached before the critical pressures predicted by elasticity** can be approached for the particular material use in the FEA analysis. Therefore, for some materials, an elastic model tends to overestimate liners' buckling resistance.

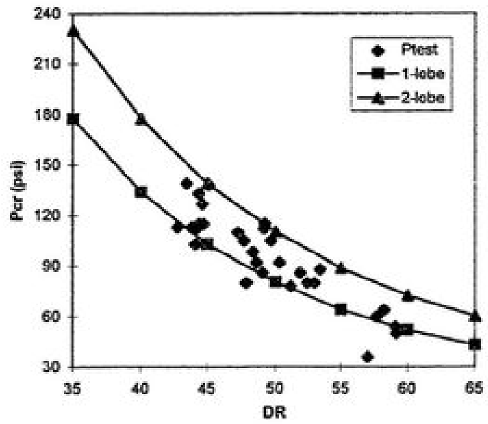

Figure 4. Comparison of predicted and observed buckling pressure from Zhao 1999.

The finite element predictions show excellent agreement with experimental data, with most of the experimental data falling between the predicted lower and upper bounds (Figure 4).

Q. Zhao concluded that the design of such liner systems should be based on one-lobe buckling models corresponding to a conservative lower-bound buckling prediction.

Effect of geometrical parameters (DR, gap and ovality) on the buckling pressure:

Q. Zhao conducted a parametric study with the FEA by varying the 3 geometric parameters DR, Gap and Ovality. An empirical analytical formula of the buckling pressure (in one-lobe mode) derived from the Glock's model was calibrated to the results.

$$P_{cr}/E^* = aDR^{-m}$$

With $E^* = E/(1 - \nu^2)$

Where coefficient a and exponent m depends on the geometrical parameters, DR, gap, Ovality and where determined by Lagrangian polynomial interpolation on FEA results.

Note that In the Glock's model where gap and ovality are zero, a = 1.0 and m = 2.2 (Glock, 1977).

Comparisons between the empirical formula and test results was good. The formula gave lower bound prediction as expected because the one-lobe mode was used.

An analytical formula for the enhancement factor K was also derived:

$$K = \frac{a}{2} DR^{3-m}$$

This formula shows that the enhancement factor increases with the DR ratio but decreases with the gap and the ovality.

According to this formula, the value of K = 7 used by default, can be overconservative for high values of DR and low gap but it can be unconservative for low DR (<50) and high gap (>1%).

D. Hall and M. Zhu (Hall, 2001) studied the contact conditions and stresses which develop in a liner under uniform external pressure.

Concerning the contact conditions, they showed that as the external pressure or time increases, the contact area increases and then the extension angle of the deflection lobe decreases. This decreases in lobe span is one way that helps the liner to resist buckling. They also showed that the lobe span is lower for thinner liners for a given percentage of the critical pressure that means that host pipe contact improves the buckling resistance of thinner liners more than thicker liners and this explains why the enhancement factor K increases with the DR, an increase observed experimentally and theoretically by Glock's model.

Concerning the stresses, they showed that imperfections (gap, ovality, longitudinal intrusion) increase the level of bending stress which quickly become dominant in comparison to hoop stress. Consequently, they suggested that flexural properties as measured in a bending test are appropriate for design cases involving longitudinal imperfections, large gaps or large ovalities, especially for liners with high DR values (> 60). Conversely, compressive material properties appear to be appropriate for liners with low ovalities and gaps with no longitudinal imperfections, especially for liners with lower DR values (< 30).

Long term behavior (creep-induced buckling)

Q. Zhao (Zhao 2001) proposed an empirical relationship relating the critical time to the ratio of applied pressure to the short-term critical pressure:

$$T_{cr} = T_0(1/PR - 1)^n$$

Where $PR = P/P_{cr}$ is the ratio of applied pressure to the short-term critical pressure.

The parameters T_0 and n are dependent on material properties (creep and plastic) and liner parameters (DR, gap, Ovality), they were determined by fitting on finite element results.

The empirical relationship gives excellent agreement with the finite element results.

Fairly good agreement was found between the predicted critical times and observed results from long-term buckling tests.

However, the determination of the parameters T_0 and n, which must be carried out for each material, is a relatively complex process which includes short-term and long-term (creep) testing, a finite element parametric study and a fitting.

Research conducted by J. Boot (University of Bradford)

Boot and Welch (1996) conducted a program of research comprising short-term testing of sprayed polyurethane lining, 10 mm thick, in circular (450 mm ID) longitudinally quartered clay pipe rendered 'rigid' with steel bands and aggregate backfill. The annular gap varied from 0% to 10%. The research following research program was undertaken:

- Determination of the constitutive behaviour of the lining material over a 50 year period.
- A series of 14 short term tests under increasing pressure to failure.
- Development of a mathematical model capable of utilising the tests results to predict performance of liner subject to long term creep under constant load.

The annular gap varied from 0% to 10%. In all but one test the two-lobe deformation mode was obtained. Note that the fluid used was air and not water, so there was no buoyancy effect which may explain the high number two-lobe modes.

Results show an approximately linear reduction in strength with increasing annular gap.

Boot and Welch (1996) developed a fully consistent finite element analysis theoretically capable of incorporating the effects of imperfect geometry and visco-elastic material behaviour. However, this approach was not suitable for routine design.

Boot (1998) incorporated in the Glock's model a small annular gap, he obtained an approximate solution of the following form:

$$log\left(\frac{p_{crit}}{E}\right) = mlog\left(\frac{D}{t}\right) + logc$$

where the parameters c and m depend on the annular gap to radius ratio and the number of lobes (1 or 2) and are pre-calculated in tables.

Note that the Boot's formula is quite similar to that the modified Glock formula used by Q. Zhao in his FEA parametric study.

Revision of the WRc SRM Sewer Renovation Design Method and a new approach to Quality Control of sewer renovation systems

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PAPER: This paper describes the changes to the WRc Sewer Renovation Design method in the 2021 edition of the Sewerage Rehabilitation Manual (SRM) publication and associated changes to the recommendations for quality control (QC) of sewer renovation work. The changes to the design method include the use of partial factors in line with the principles of EN 1990 (CEN, 2002), updates to traffic loading, calculation of bending moments and bending stress in Type I design and a new method of calculating the buckling resistance of liners. In line with the EN 1990 principles the revised SRM recommends a statistical approach to QC that takes into account the implications of using short-term test data for assessing the long-term properties of the materials.

1. INTRODUCTION

The first edition of the Sewerage Rehabilitation Manual (SRM) (WRc, 1983) was one of the earliest published design methods for sewer renovation systems and followed an extensive programme of research. The method is used both in the UK and internationally. Although the SRM has been revised on a number of occasions since then, the sewer renovation design method is largely the same as it was in 1983. Since that time there has been further research into the structural design of renovation systems, and improved materials. In addition, structural design generally has adopted a limit state design approach in line with the principles of EN 1990 (CEN, 2002).

The SRM considers two design cases. Type I, in which the liner forms a composite structure incorporating the host pipe, the liner and the grout which interacts with the soil to take the soil and traffic surcharge loads. Type II in which the host pipe, interacting with the soil, continues to take the soil and traffic surcharge loads, but is stabilized by the liner, that is only designed to take the hydrostatic load from the groundwater.

For both methods of design, it is important that the installed materials, many of which are cured in situ, achieve the long-term properties specified in the design. Product standards, which once specified minimum values for material properties now typically require manufacturers to specify 'declared values' for material properties. The standards do not explicitly state the basis for the declared values and the test methods typically require the mean value to be reported, with the standard deviation if requested. This has important implications both for design and when short-term quality control test results are evaluated following installation.

2. LIMIT STATE DESIGN

The revised method adopts the following principles from the Eurocode design principles as set out in EN 1990 (CEN, 2002).

1) The use of Characteristic Values of actions (loads) and material properties based on fractile values from the distribution of values.

2) The use of partial factors applied to key actions and material properties in place of a single factor of safety.

Where the lower limit is the worst case, the characteristic values for material properties should be based on the 95% fractile value. This is the value which is exceeded in 95% of cases This is illustrated in Figure 1.

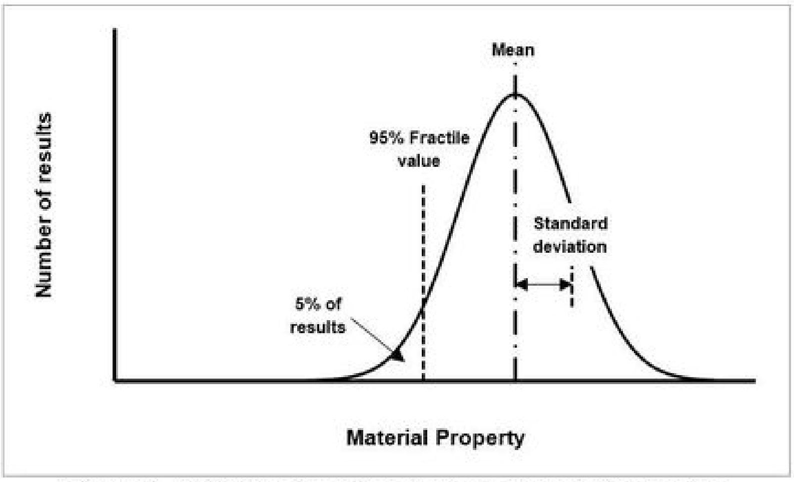

Figure 1. 95% fractile values from a normal distribution

In the few cases where the upper limit is the worst case, the 5% fractile value should be used.

For the elastic modulus EN 1990 (CEN, 2002) recommends using a mean value rather than a characteristic value except for instability limit states. One of the key limit states in liner design is buckling, which is an instability limit state. For this limit state the 5% fractile value is therefore recommended rather than the mean value which is used for other limit states.

Manufacturers' declared values for most material characteristics should therefore be the 5% fractile value. Additionally, a mean value of elastic modulus should be specified for use in calculations other than buckling. In whatever case it is now imperative the manufacturer states the basis for their declared values.

Partial factors for actions are in accordance with the recommendations in BS 9295 (BSI, 2020). Partial factors for material properties are taken as 1.2 (for factory produced linings (e.g. close-fit liners). For site cured materials (e.g. CIPP liners) a value of 1.5 is recommended for short term properties and 1.67 for long-term properties. These align with practice in France (Thépot et al. 2016).

3. TYPE I DESIGN

There have been four key changes to the soil and traffic surcharge loading calculations for Type I liners. The traffic loadings have been updated in line with EN 1991-2 (CEN, 2003) and the load distribution method aligns with BS 9295 (BSI, 2020) which allows users to consider alternative load cases (e.g. railway loadings) by reference to that standard.

The calculation of the bending moment has been changed. This now assumes the composite structure will act in a similar way to a new pipe. For circular liners, the moment is calculated using the method developed by Moore and Doherty (Moore & Doherty, 2016). For 3 x 2 egg-shaped sewers BS 9295 is used to calculate the bending moment.

Where the liner has a relatively low elastic modulus the strain at the boundary between the liner and the grout can exceed the capacity of the grout before the working stress in the liner is reached. If this happens then the renovated sewer would cease to act as a single composite structure. An additional check is therefore included for this eventuality which will have the effect of derating the strength of some types of liner material.

To facilitate this the second moment of area of the composite structure and the position of neutral axis are calculated using the transformed area method as proposed by McAlpine (McAlpine, 2005).

4. RESISTANCE TO BUCKLING

The buckling resistance of circular liners, whether under the long-term hydrostatic load or short-term loading during grouting, was previously calculated based on the Timoshenko equation (Equation 1) for unrestrained linear buckling theory (see Figure 2). This method is also used in North America (ASTM, 2009).

$$\boldsymbol{P_c = 2 \cdot E \cdot \left(\frac{D}{t}\right)^{-3}} \qquad [1]$$

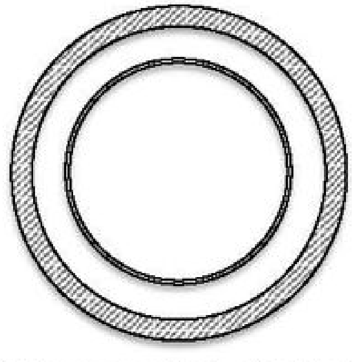

Figure 2. Liner unconstrained by the host pipe

In previous editions of the SRM, the Timoshenko equation was modified by an enhancement factor of 7 to take account of the support provided to a liner restrained by the host pipe. This was based on the work of Aggarwal, & Cooper, (1984).

Figure 3. Liner constrained by the host pipe

An alternative approach which better represents the case where the gap is small has become more widely used in Europe (ASTEE, 2014, DWA, 2015) is to use the Glock equation (Equation 2) (Glock, 1977) for fully restrained buckling of circular liners (see Figure 3) and apply a reduction factor.

$$\boldsymbol{P_c = E \cdot \left(\frac{D}{t}\right)^{-2.2}} \qquad [2]$$

Gumbel (2001) compared the Aggarwal, & Cooper, (1984) test results with both these approaches (see Figure 4). This showed that the Glock approach gives a better fit to the data than the modified Timoshenko approach. The scatter is explained by the gap imperfections which neither method considers.

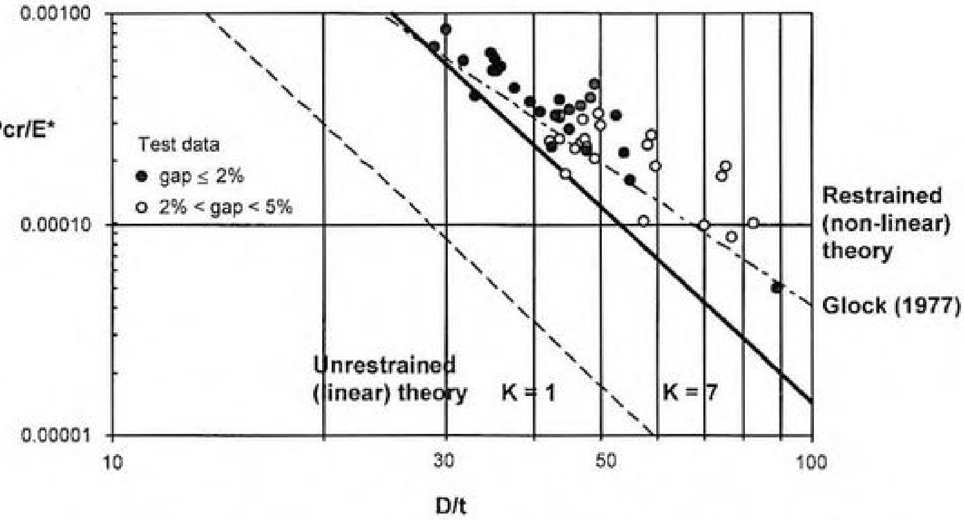

Figure 4. Aggarwal's 1984 buckling data compared contemporary theories (source Gumbel (2001))

For long-term buckling of circular liners, the SRM now uses the implementation of this by Boot (Boot, 1998). A gap reduction factor is based on the subsequent paper by Boot (2005) and using data from a finite element analysis by Boot, it was established that the ovality factor used in the original SRM could be applied to this new approach.

For short-term buckling during grouting, however, the gap is relatively large in order that grout penetration can be assumed and in this case the liner is considered unrestrained, and the Timoshenko equation is used without an enhancement factor.

For non-circular liners the method developed by Boot (Boot et al, 2014) is used to calculate the critical buckling pressure and the tensile stress in the liner during buckling. Design charts have been developed to apply this method.

5. QUALITY CONTROL

For quality control purposes it is important to ensure that an installed liner will have the required long-term material properties. For polymeric materials these are typically evaluated using a 10,000-hour test with the results logarithmically extrapolated to a notional 50 year value (see Figure 5).

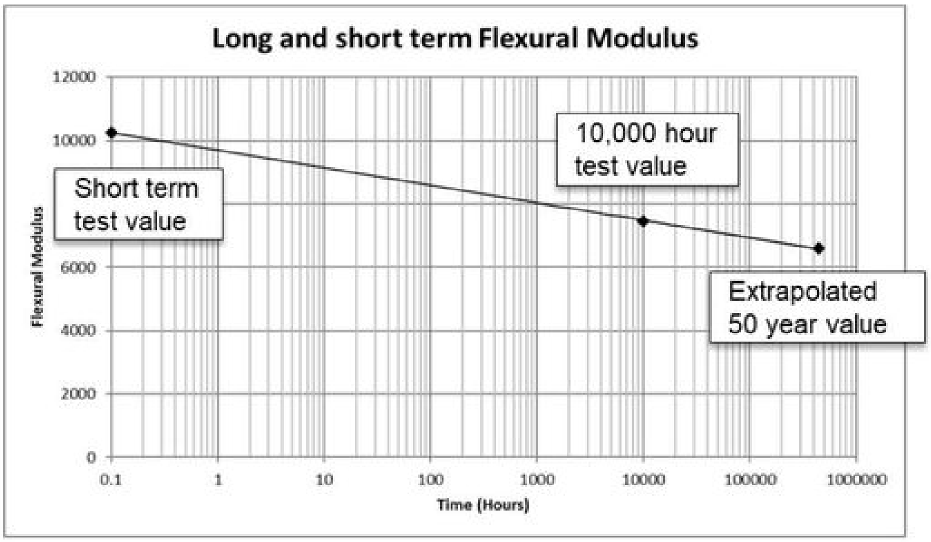

Figure 5. Evaluation of long-term material properties

Clearly it is not practicable to use a 10,000-hour test for quality control purposes and so the ratio between the short- and long-term test results obtained during type testing are used to calculate a conversion factor which is assumed to be constant. This is a reasonable assumption provided the material is the same as that tested during the type testing. In the future it would be useful to develop a test to establish the state of cure of such materials, perhaps similar to the β factor used in assessment of conformity of GRP pipes in CEN/TS 14632 (CEN, 2012)

However, if, in good faith, a manufacturer decided to declare a value somewhat less than the true 5% fractile value – to allow a further factor of safety, then it is possible that a test result that exceeded the declared value was still incompletely cured and, in this case, it is questionable whether this conversion factor for the long-term property was still valid. It is therefore essential that manufacturers are transparent in the derivation of their declared values and that both the long and short-term values should be based on sound statistical principles using the same approach.

Another question arises from the use of 5% fractile characteristic values. Since only 95% of values are expected to exceed this value, it is quite possible that a small proportion of test results less than the characteristic value are still consistent with the results from the type testing.

A more rigorous approach to assessing the results from quality control testing is therefore required that tells us that the material is the same as the type-tested material and takes account of the expected variation in the test results.

To achieve this, it is proposed that the manufacturer should declare, not only the long- and short-term 5% fractile characteristic values, but also the mean and standard deviation of the results of the short-term type tests. The user can then use standard statistical tests to check that the mean and standard deviation of our QC test results are consistent with the mean and standard deviation of the manufacturer's test data to the required degree of confidence.

6. CONCLUSIONS

The SRM design method has been thoroughly updated taking into account the latest research in sewer renovation design and aligning with the limit state design approach in line with the principles of EN 1990 (CEN, 2002).

Manufacturers' declared values need to be consistent with the design method used and for the SRM method, characteristic values should be 5% fractile, and in some cases 95% fractile characteristic values. The inclusion of undocumented additional safety factors is detrimental to the understanding of quality control results and could give users a false sense of security.

To properly monitor the quality of installed liners, a statistical approach should be taken which requires manufacturers to publish the mean and standard deviation of their short-term test results.

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External pressure tests on large diameter jacking pipe system

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1. INTRODUCTION

The planned Isola Farnese – Crescenza Sewage system project in Rome includes more than 5 km of DN2000 glass fibre reinforced (GRP) jacking pipe. The system will not be pressurised, but with burial depths of up to 60 m, the external water head will be as high as 26 m. This requires that all joints and grouting nozzles be able to withstand that pressure. The testing and qualification of the joints, together with a partial case history of the project, is the topic of this paper. The test requirements were a leak-tight seal during a 24 hour test at 3 bar.

To ensure that the system can take these pressures, a full-scale testing program was conducted in Norway, where the system was taken to its limits. Due to travel restrictions between countries, the witnessing of the test by the customer was accomplished by online life streaming the tests.

The owner of the project is the utility company ACEA, the design engineer is LaboratoRI SpA, the contractors are I.CO.P. Spa and Mario Cipriani S.r.l. The tunnel boring machine is a Herrenknecht AVN. The pipes are centrifugally cast Hobas GRP pipes, manufactured by Amiblu Poland Sp. z oo. in Dabrowa Gornicza.

2. PROJECT DESCRIPTION

The ancient village of Isola Farnese is located on a hill between the Storta valley and the San Sebastiano valley about 7 km North-East of the Grande Raccordo Anulare (Il Raccordo), the A90 motorway that encircles Rome. The name comes from Cardinal Farnese, who bought the village, castle and all, from the Orsini family in 1527, and furthermore, it is built on a hill surrounded by waterways and an artificial moat which isolates it from the surrounding countryside. Along with the neighbouring villages it is of archaeological importance and the whole area is a peaceful, green lung, just outside the city proper.

The project is part of the Isola Farnese – Crescenza sewage collector to the treatment plant and consists of 5325 metres of DN2000 pipes and 90 m of DN1200 pipes, overall length over 5.4 km, with both straight and curved sections. Pipe lengths are 6 m for both. A total of 6 jacking sections are planned, lengths between 102 and 1235 m, with 41 planned intermediate jacking stations. The highest jacking force is estimated 8500 kN. Depth above pipe crown is between 3 and 60 m, with highest ground water level at 26 m above the invert. The nominal pressure of the pipes is 1 bar.

A plan view of the project is presented in Figure 1, showing the location of jacking pits.

Figure 2 shows the profile of the topography and the planned pipeline. The water flow is from north-west to south-east, while the jacking course is in the other direction. The jacking starts at a relatively shallow cover of 2 – 5 m, with a jacking length of 102 m between pits H and G. From pit G to F the jacking length is 501 m, with cover depth of 5 -10 m. The next section is 932 m, with a cover of 10 - 44 m, and thereafter, from pit E to D, the jacking length is 640 m with cover 16 – 23 m. The second longest jacking section is 1050 m from pit D to C, with cover depts 20 – 60 m, and then comes the longest section, 1235 m, with cover depths 2 – 59 m. In these locations the ground-water level is also at its highest, estimated at 26 m over the pipe.

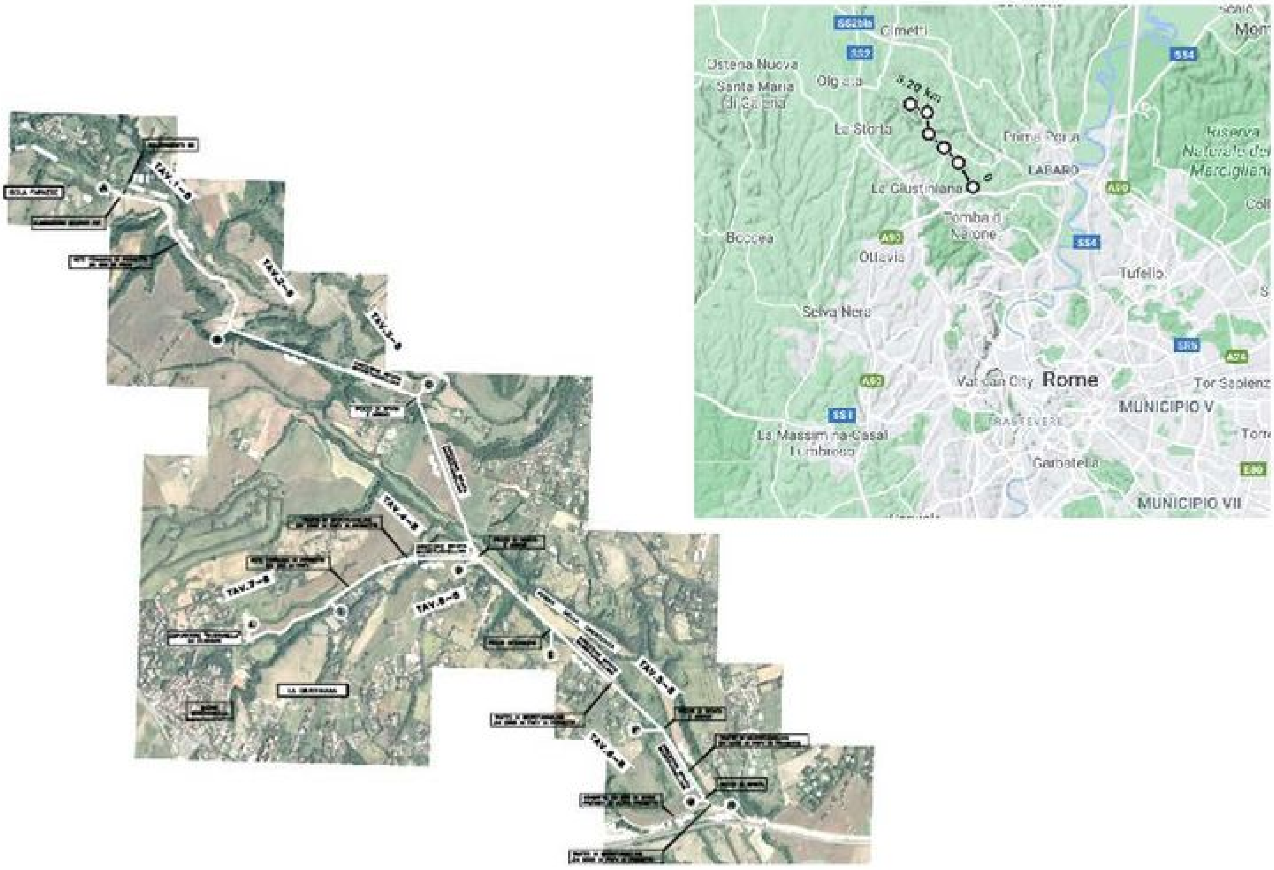

Figure 1. Location and plan view of project (background: Google maps)

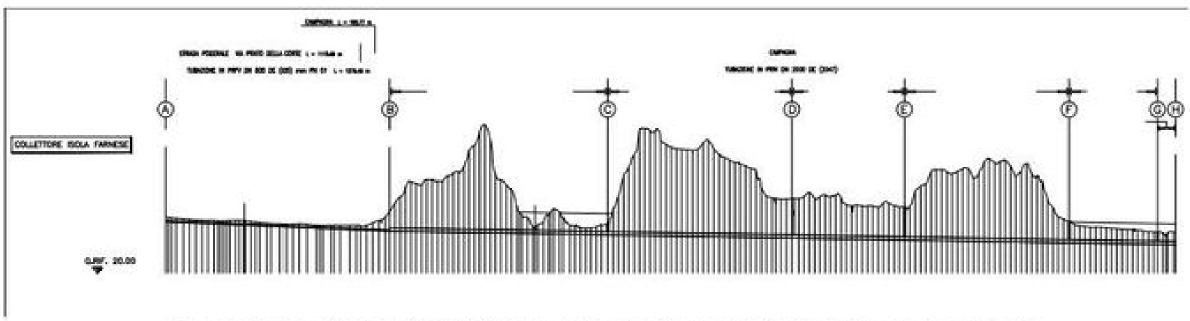

Figure 2. Profile for DN2000 pipe, showing topography and location of shafts

3. JOINTS

The joints between pipe sections are of the Hobas PJC type PN6, made of stainless steel with a continuous rubber profile with two sealing lips on each side (see Figure 3). When used with a rebated pipe spigot it is flush with the pipe outer diameter, which is essential for the jacking operation. The ends of the coupling are bent inwards; in addition to providing increased stiffness this prevents dirt from entering the sealing area during the jacking operation.

The joint is designed for 6 bar internal pressure, with the sealing lips pointing towards the spigot end, which facilitates assembly. The sealing principle is based on the internal water pressure contributing to the gasket sealing with the lips being pressed against the pipe spigot. Angular deflection and misalignment due to shear can also be accommodated.

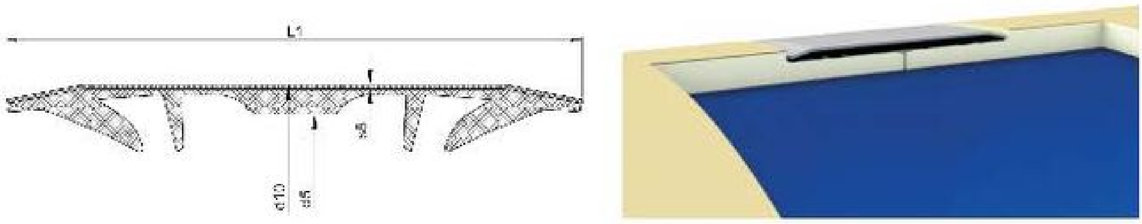

Figure 3. Pressure Jacking Coupling (PJC) cross section and assembly

The joint seals also against external pressure, although it's not specifically designed for it. The purpose of the test was to verify that it will seal against the pressure from the high ground water, and by what margin.

4. TEST PREPARATION

The test was performed at the Amiblu Technology laboratory in Sandefjord, Norway. The laboratory specialises in testing glass-fibre reinforced plastic pipes and associated fittings and components. It has a capacity to test up to 700 bar pressure, and diameters up to 3000 mm. The laboratory has been accredited according to ISO 17025 by Norwegian Accreditation since 2003.

The pipe samples were manufactured by Amiblu Germany GmbH, Trollenhagen, in February 2020. The PJC coupling was manufactured by Ritec. The samples were received in the lab, with the coupling mounted on one pipe, as shown in Figure 4.

Figure 4. Test pipe with mounted PJC coupling

The method for applying the external pressure was fitting a second coupler on the outside, covering the whole flush steel coupling (see Figures 5 and 6), creating an annular space between the two. The coupler is a specially designed GRP sleeve, with two rubber rings positioned in grooves in the external coupler, sealing against the outside pipe wall on each side of the PJC coupling.

Two ½ inch valves were fitted into the external coupler to holes drilled through the sleeve wall. A small steel plate was welded to each fitting, prior to gluing it in the hole. A few layers of chop strand mat were laid over to secure the fitting in place.

One of fitting was placed close to the invert for filling and pressurising, the other was placed at the crown for air venting and for the pressure gauge.

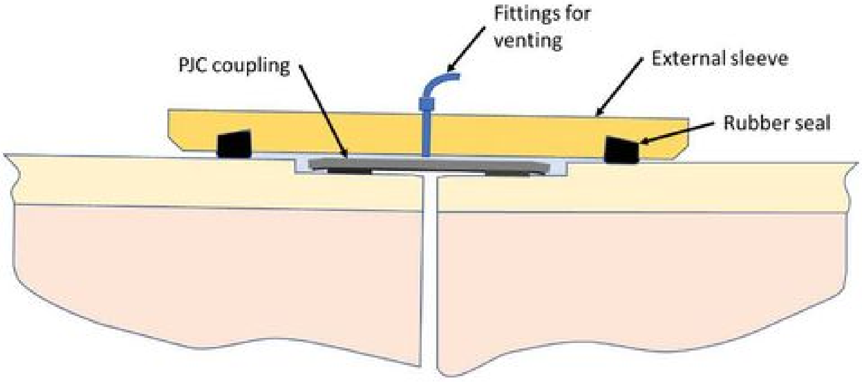

Figure 5. Method for applying external pressure on joint

To meet the project specific requirements an angular deviation and a draw between the pipes needs to be maintained during the test. The specified angular deviation is 0.15°, while the draw is specified as 0.2% of the 6 m pipe length, or 12 mm.


Figure 6. External sleeve for pressure application

The three photos in Figure 7 show the joints being assembled. To ensure proper positioning and orientation of the gaskets, the external sleeve is first slidden over the pre-mounted PJC coupling and then the other pipe is inserted into both; first the spigot has to fit into the PJC coupling, and then the outer sleeve must meet the outside of the pipe wall without dislodging or shearing the gasket.

After assembling, the specified angular deflection and draw need to be imposed. This was achieved by placing shims of varying thickness at intermittent locations around the circumference. Come-along jacks were then used to pull the two pipes together. During the test, these jacks were engaged to maintain the specified angular deflection and draw.

After assembly, the water pump and pressure gauge were connected, and then the joint was ready for testing.

Figure 7. Assembly and adjusting angular deflection

5. PRESSURE TEST AND RESULTS

The original plan called for the test being witnessed by representatives of the owners and the engineering office in March 2020. Due to the outbreak of the world pandemic, which had already hit most countries in Europe at that time, other means were resorted to.

Four video cameras were placed at strategic locations inside and around the test sample, with the lenses aimed at the positions where potential leaks would be detected, as well as at the reading from the pressure gauge and the chronometer. This set-up is shown in Figure 8. For the duration of the qualification test the view from these cameras were streamed online over Youtube, to be watched in Rome by the parties involved. This turned out to be a satisfactory solution, providing a long and exciting, albeit not an eventful, moving picture, where most of the time, the only motion was the clock ticking.

The test itself consists of applying and maintaining the water pressure in the annular space between and PJC coupling and the external GRP sleeve. The pressure was recorded every second.

Because of the relatively low water volume, minute changes in temperature and creep in materials affect the pressure considerably. To maintain the pressure within acceptable limits an actuator was connected to the system through a small orifice fitting.

Figure 8. On-line streaming of test, day and night

Three separate tests were conducted: first a 24 hours 3 bar test, which was the specified requirement. Then the pressure was increased to 6 bar and maintained for another 24 hours. Finally, to explore the limits, the pressure was increased to 16 bar for 15 minutes.

Figures 9 – 11 show the recorded pressure versus time for the three tests. At the beginning of each test a fluctuation in pressure is noted, but as time passes the system stabilises.

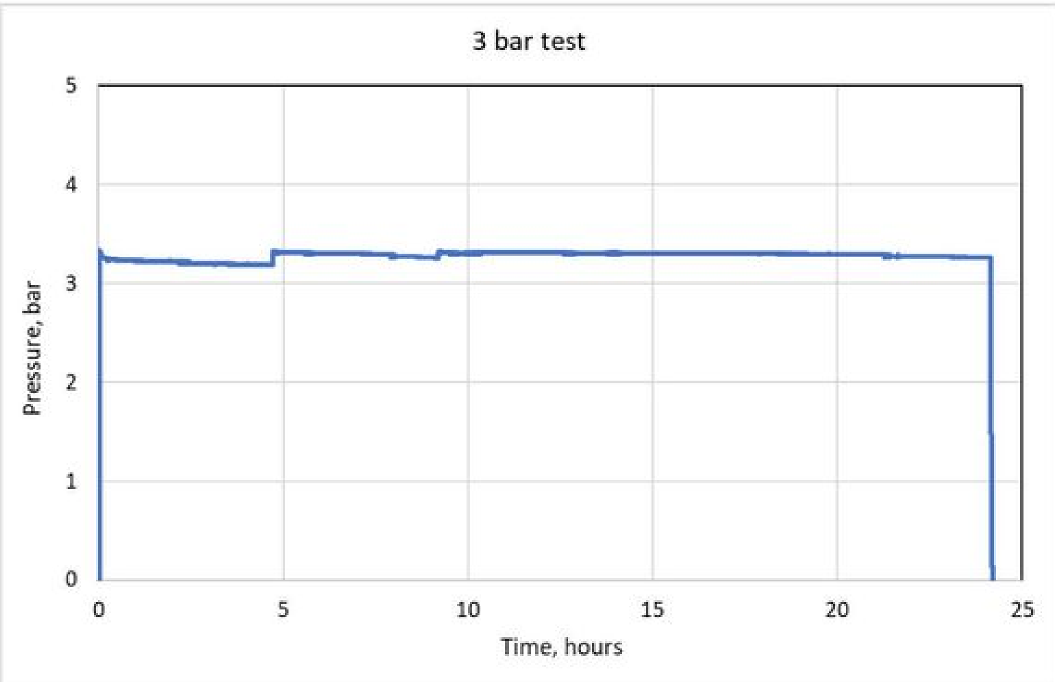

Figure 9. Twenty four hours 3 bar test

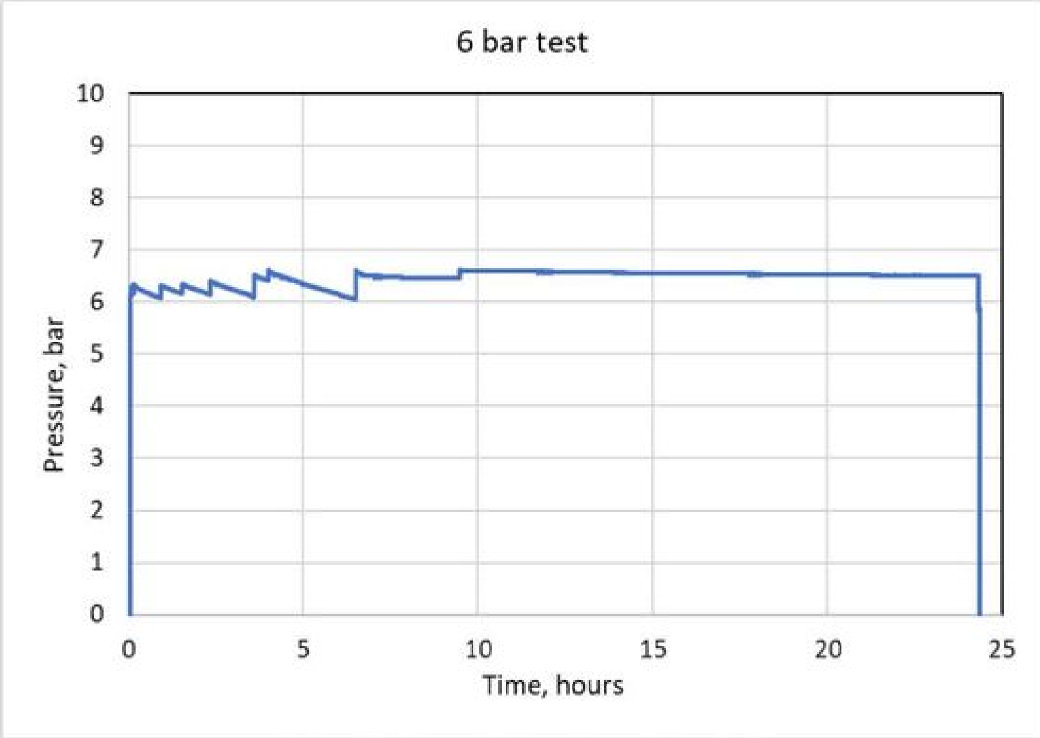

Figure 10. Twenty four hours 6 bar test

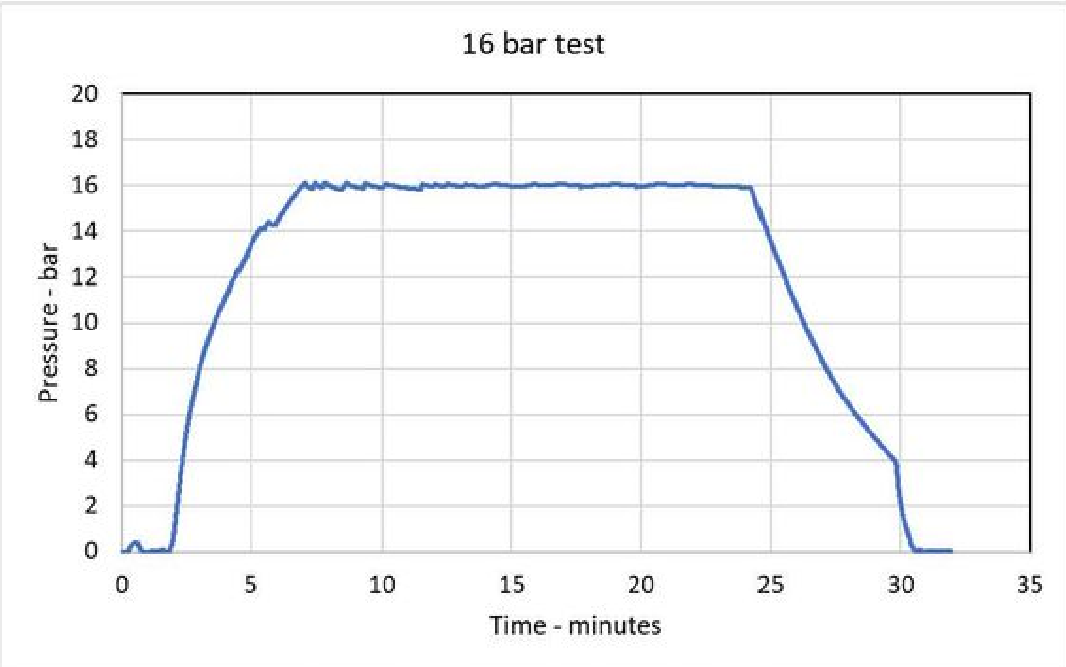

Figure 11. Fifteen minutes 16 bar test

No leaks were detected during any of the tests.

6. PROJECT PROGRESS

Figure 12 shows three photos from the site, two jacking pits and an intermediate jacking station. At the time of writing, approximately 2,2 km of DN2000 pipes have been installed, including the longest drive of 1235 m. It is estimated that the jacking will be completed in the first quarter of 2022.

No major issues have yet surfaced.

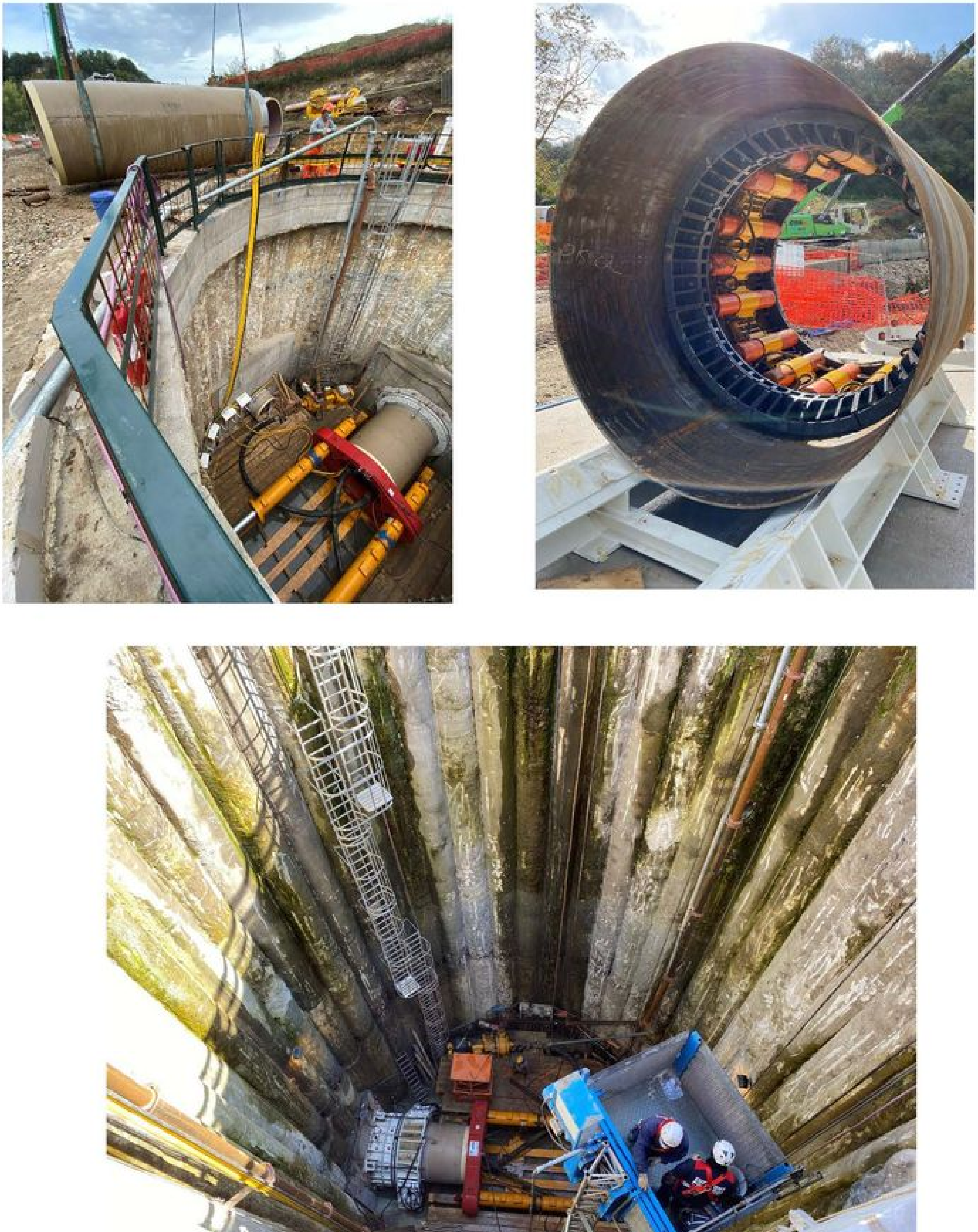

Figure 12. Jacking process, from top left: pit 2, intermediate jacking station, pit 3 (bottom)

"Real-time monitoring of UV emitters as requirement for controlled and homogeneous curing of large diameter liner with big wall thicknesses"

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RelineEurope GmbH

1 Introduction

Anyone overseeing a production process involving UV-light curing understands the importance of monitoring and validating that the process is working within the specs.

There are a variety of UV curing light measurement systems on the market that were designed specifically to monitor the light output from UV sources used for curing.

Some were optimized for cost and may lack performance and quality. This may include linearity and traceable/repeatable calibration, for example.

There are many designs of both photo initiators and UV-light sources in use, making designing a measurement system to work universally across platforms challenging. Add to this the fact that there is no industry standard stating which wavelengths you should measure, or how the spectrum should be weighted, or at which wavelength the system should be calibrated. In short, light measurement system designs vary greatly by manufacturer, leaving many unanswered questions for end-users in selecting the right measurement system for their application.

UV light-curing for CIPP hose liners have been used as standard since the early 1990s. Whereas they were initially mainly used only in so-called small diameters up to DN 500 for the rehabilitation of drainage lines, these hose liners, especially in recent years, also offer excellent solutions for the remediation of large pipe diameters up to DN 1900.

The liners with their different diameters and wall thicknesses of up to 20mm should harden completely, evenly, and efficiently with UV-light curing systems. The UV-light curing takes place in fractions of a second. In order to implement this, a controllable and reproducible curing process must be ensured.

2 UV-light curing of hose liners (UV CIPP)

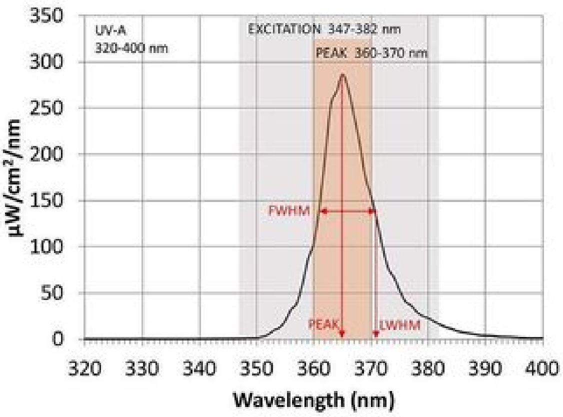
UV radiation is used for the hardening of unsaturated polyester and vinyl ester resins. The UV-light curing in hose liners is consistent and easily controllable regardless of humidity and temperature. It is therefore very advantageous for individual job site conditions as well as for long transport routes.

The advantages of this curing process are obvious. However, there are many influencing factors that need to be monitored to ensure a controlled, reproducible curing of the respective liner thicknesses.

These include:

a. UV-emitter performance under the following conditions:
   i. light source geometry (position of the single UV-emitter),
   ii. distance of the UV-emitter from the liner surface,
   iii. homogeneity of irradiation intensity,
   iv. transparency of hose liner materials - low light absorption
b. UV spectrum in combination with UV-light photo initiators
c. Cable length

For an effective cure the UV-light source wavelength has to match the photo initiators' absorption peak, which can range from 200 to 450nm depending on the specific product and application. Short wavelengths are often selected for thinner surface applications, to allow faster throughput speeds. Longer wavelengths are absorbed into the substrate and offer greater depth penetration for thicker materials.



Measurement Results	
Peak Wavelength:	365 nm
FWHM:	10 nm
LWHM:	371 nm
Excitation:	3,051 µW/cm²

Figure 1: UV Spectrum of different wave lengths

There are two main **units of measurement** for curing: **UV-intensity (or irradiance)** expressed in W/cm2 (watts per square centimeter), and **Dose** or **Exposure** expressed in J/cm2 (joules per square centimeter).

UV-intensity is the amount of light reaching a surface. Higher UV-intensity is often important for thicker substrates, to ensure the output is capable of penetrating deep enough to allow a complete cure. Longer wavelengths of UV are often measured when cure depth is important, as longer wavelengths are absorbed into coatings for deeper penetration.

Exposure is the summation of UV-intensity over time, using the formula (W/cm2 x seconds). It is often referred to as the integrated irradiance. This total of the UV exposure during the curing process is a critical measurement for all UV-emitters, photo initiators, and UV-light sensors. It accounts for variations in the intensity during a measurement cycle, and over long periods of time.

As the intensity of the UV-emitter decreases with age, it is often possible to extend the life of the UV-emitter by monitoring both the peak irradiance and total exposure time by reducing the curing speed during the process, thus saving on premature and costly UV-emitter replacements.

3 UV-light hardening process

In a UV-light curing process, existing chemical chain building blocks are crosslinked by UV radiation-induced polymerization. In a fraction of a second, the networked system is dry (cured) and abrasion-resistant and can be processed immediately.

An important further prerequisite for UV curing is the adaptation of the used wave spectrum to the used photo initiators, i.e., it is not necessary to use any UV emitters that are not adapted to the respective light initiators in the resin system. For thin and especially thicker layers, as they occur with the hose liners, the curing takes different lengths depending on the performance of the UV-emitters used, the geometry of the light source, the distance of the UV-emitters to the liner surface and the transparency of the hose liner materials.

3.1 UV-emitters and their properties

The necessary UV-emitter is a gas discharge lamp and is used to produce the necessary UV-radiation. The high-pressure UV-emitter is not made of fluorescent-coated normal glass like fluorescent lamps or fluorescent tubes, but of clear quartz glass in order to make the necessary UV-radiation for curing directly, without further, significant light absorption, hit the liner material with the maximum possible UV-radiation intensity.

The UV-light curing systems offered on the market offer different UV performance. Different UV spotlights from different manufacturers are used.

The nominal power ranges from 400W to 4000W, especially UV-emitters with 400W, 600W, 1,000W, 2,000W, 3,000W and 4,000W.

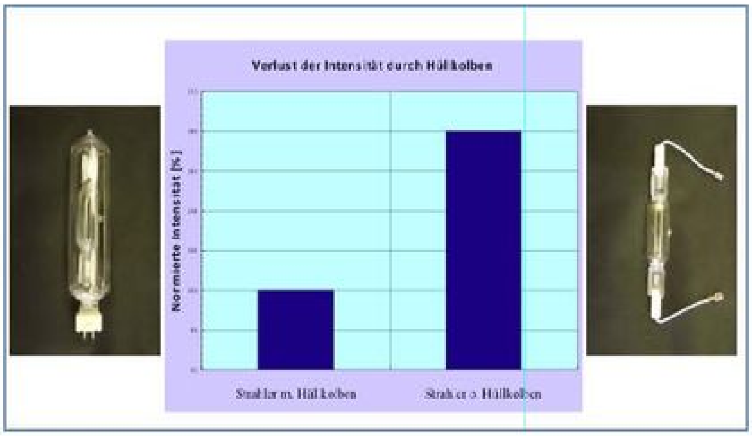


Figure 2: UV-emitters of different performances

Newly developed crystal-glass UV-emitter with xenon filling and platinum finishing have an output of up to 4,000 watts, making them the most powerful light sources offered for UV-light curing. The quartz crystal glass used ensures the lowest light absorption, a platinum finishing allows a faster ignition of the single UV-lamps (source: RELINEEUROPE).

In recent years, more and more manufacturers have been offering different UV-light sources for curing hose liners. At first glance, there are a wide variety of constructions. It is not detectable whether these were matched in conjunction with UV-light curing hose liners to be hardened. Nor information is available concerning the suitability of the used photo initiator additive in the resin for matching efficiently with the UV-light spectrum. In some cases, there are very different distances to the liner surface.

The valid physical UV-light drop/distance law applies:

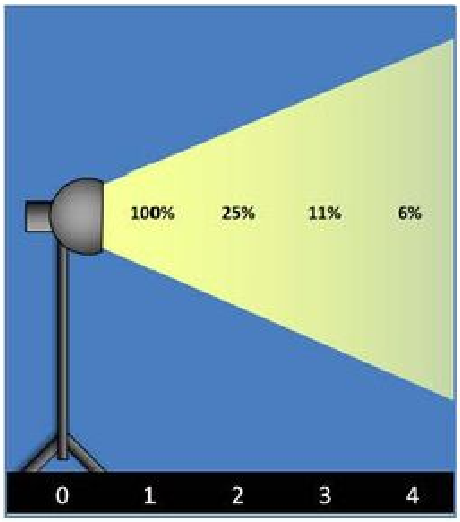


Figure 3: Drop of UV-light intensity

- **The UV-light intensity is inversely proportional to the square of the distance.**
- When the distance from the UV-emitter is doubled, the UV-irradiation must cover four times the area or is reduced by a factor of 4. If the distance triples, even 9 times the area need to be irradiated.

3.2 Performance of UV-emitters

In addition to the UV photo initiators, some hose liner manufacturers add additional thermic-curing initiators, so-called "peroxides", to their UV-light curing resin system to cure even larger liner thicknesses over 7 - 8mm with the common UV-light systems available in the market.

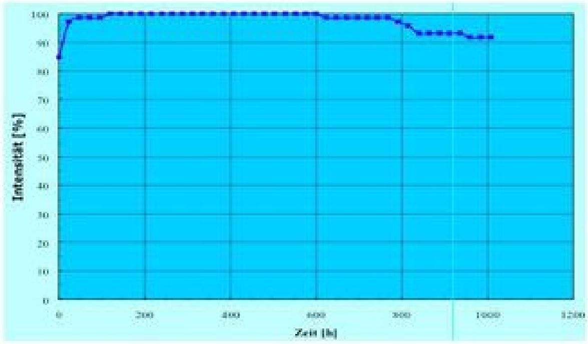


Figure 4: Performance drop due to ageing

The monitoring and controllability of the so-called combination curing is of particular importance due to the different UV-light curing systems used, since any temperature effects trigger the hardening process, but without using the UV-light curing.

The service life of a UV-emitter is defined by the physical condition and the operation service life (runtime). The runtime depends on the operating hours and the frequency of the ignition circuits and varies depending on each single UV-emitter and manufacturer. The manufacturer is responsible for defining the useful service life. The aim must be to make the UV-intensity at the end of the service life still sufficient, with the predetermined curing speed, to harden the hose liners with their respective wall thicknesses.

UV-emitter, like almost all other lamps, age with the runtime in the output power and should therefore be changed regularly. A permanent check of the respective individual UV-emitters in the UV-light curing systems is necessary. Ideally, the performance check of the individual UV-emitter should be carried out during the regular servicing of the entire UV system (generally at the latest every two years).

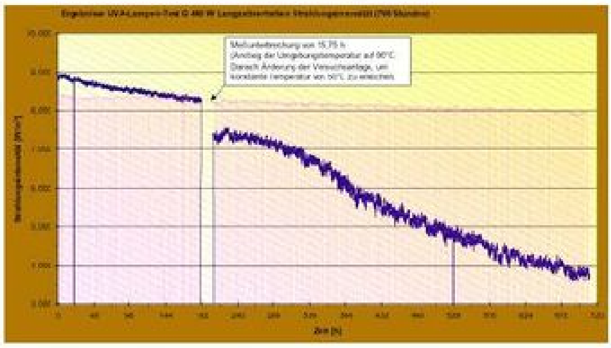


Figure 5: Performance drop due to operation life

However, it is essential that a sufficient UV-intensity is provided by the irradiation sources to allow safe curing of the lining hoses. Depending on the running time of the UV-emitter, it may require more than one exchange per season. Most manufacturers specify an hourly number for which they guarantee that the UV-emitter will not lose power! A loss of performance on individual UV-emitters has a negative effect on the curing, as the curing speeds specified by the hose liner manufacturers can no longer be used. The result is a incomplete hardening of the resin system.

With suitable sensors and measuring instruments, UV-intensity can be made measurable to ensure quality assurance. In order to guarantee a complete and homogeneously curing of the hose liners, the condition of the UV-light source (no absolute value) as well as the UV-emitter aging should be checked at any time via a simple relative measurement. The percentage of aging compared to the new state can thus be controlled. For reproducibility, a fixed and always consistent measuring position should be ensured.

An important prerequisite for the reproducible curing of the liner wall thicknesses is the constant UV-irradiation with almost uniform UV-power flow. In addition to the regular manual checks of the single UV-emitters, the use of Electronic Ballasts (EB) is the very best way to monitor it in real-time. There are significant advantages compared to conventional ballasts. For example, this technology allows a control of the real power output of the UV-irradiation energy of each individual UV-emitter. Each UV-emitter has an individual power control device (EB) for controlling the electrical power output, which measures the recorded actual power of the UV-emitter and compares it with the target parameters. In the event of a fall in the target parameter, the used Electronic Ballast will automatically increase the power output. This ensures that the performance of each UV-emitter, regardless of its aging process, is always constant. The predefined curing speeds of the respective UV-light sources are therefore always valid.

Electronic Ballasts also enable the optimal monitoring of the UV-emitters and their power/control. Thus, different performance requirements can be ignited with the use of a UV-emitter. For example, a 1,000W UV-emitter ignites 300W, 400W, 500W, 600W and 1,000W to harden different diameters with different liner thicknesses in a controlled manner. For large diameters, the increasing distance to the liner surface is compensated by higher UV-emitter performance: with the 2,000W UV-emitter, outputs of 1,000W, 1,500W and 2,000W, the 3,000W UV-emitters 1,000W, 2,000W and 3,000W as well as the 4,000W UV-emitters 2,000W, 3,000W and 4,000W. Thus, the distance law/light drop is perfectly considered. (Source: RELINEEUROPE)

3.3 Real-time performance monitoring of the UV-emitter

Generally, the properties and performance of UV-emitter should be tested and controlled by any manufacturer of UV-light curing hose liners or manufacturer of UV equipment. All results in the manual as well as in the respective curing speeds for UV-light sources depending on diameters or liner thicknesses.

Standard has been the automated curing protocol for every UV-light curing system for 20 years, which shows and documents various hardening parameters online. However, they still date back to the time when UV-light curing of hose liners was used especially in the range of smaller diameters up to DN 500 and with lower liner thicknesses up to approx. 6-8 mm. In order to cure larger diameters up to DN 1900 and higher liner thicknesses over 10 mm reproducibly, in addition to the control of the respective hardness parameters, the performance of the UV-emitters must be permanently checked and real-time monitored in order to detect any deviation in the curing process of the hose liner, curing problems prematurely and, if necessary, counteract them.

UV-emitter monitoring systems for different applications are commercially available. However, the selection of the proper monitoring equipment can significantly impact how efficiently the irradiance is controlled. An improper selection of the UV-sensor system can lead to the inability of the user to control the output of the UV-related process. This is even more important when the output of the process cannot be tested in real time (i.e., hardening of the hole liner thickness).

The UV monitor works by using a sensor to monitor a single or multiple lamps.

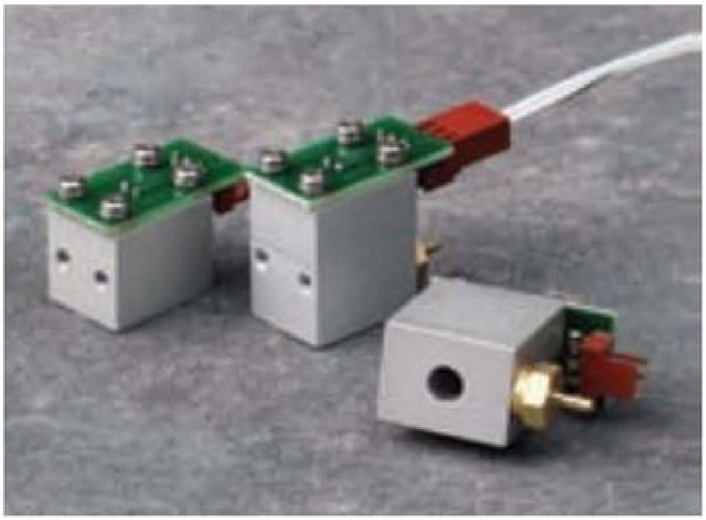


Figure 6: UV-sensor for real-time monitoring of UV-intensity

When used in conjunction with lamp status and run time indicators, the UV monitor provides an indication of how the UV lamp is performing. This performance can be provided in a 0-100% reading or in Wm/2.

Real-time monitoring and documentation within the UV rig during the hardening procedure makes this process traceable. The curing protocol is therefore an important document for QC/QS and must be included always to the jobsite documentation.

Example:
RelineEurope GmbH relies on a permanent automatic inspection and follow-up based on special electronic control systems based on the ELECTRONIC system developed in-house, which permanently measures and regulates any UV-intensity power drop.

The curing protocol shows and documents the real-time power output of each individual UV-emitter. The operator thus has the permanent possibility on real-time to detect irregularities and, if necessary, to take counter measures.

If, depending on the diameter, UV-emitter with different performances are used to harden hose liners, this is the only way to control in real-time the performance of the UV-emitter and thus ensure that the curing is carried out according to the specifications.

The additional curing monitoring is based on the control of the reaction temperature of the resin used, measured against the inner wall of the hose liner. This is done by measuring reaction temperatures of the resin (heat radiation emitted from the liner surface) by non-contact infrared sensors and displaying them on a monitor in the control panel. At least 3 infrared sensors are installed in the UV-light sources and permanently transmit the real-time measured temperatures from the liner surface to the control panel. The at least 3 IR sensors should be installed in the light chains, distributed in sequence between the elements with the UV-emitter, so that the measured temperatures can be set in relation over the curing length. Temperatures, measured by infrared sensors, built into light cores instead of light chains, allow virtually no curing control, because the sensors follow each other too closely and temperatures are detected only selectively in non-consecutive places.

The measured temperatures only indicate the reaction temperature on the liner surface. However, this does not give a reliable statement about the reaction in the rear part of the liner thickness and therefore cannot be used as a curing control.

This type of hardness control dates back to the 1990s and can no longer be used for large diameters with big liner thicknesses. At that time, only 400W UV emitters were used.

By real-time monitoring and recording all other relevant parameters, in particular the position of the UV-irradiation source, its curing speed, temperatures and pressure inside the lining hose, the curing process is fully documented.

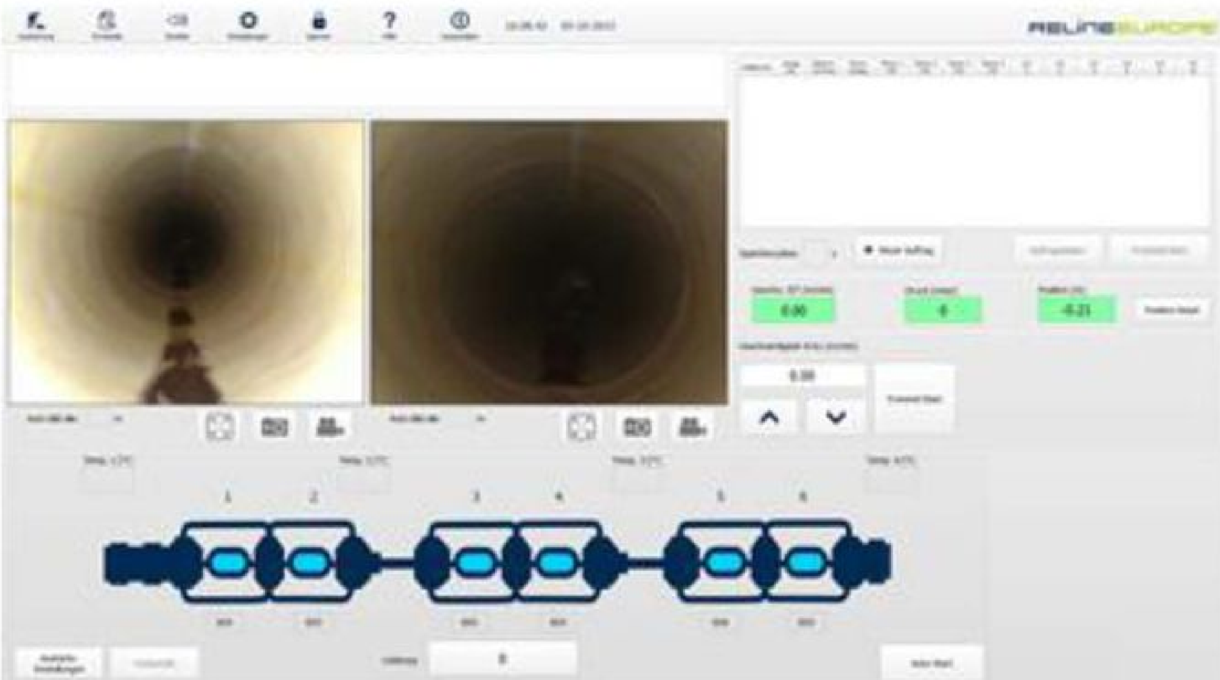


Figure 7: Real-time monitoring of the whole curing process

Figure 8 following page: Curing protocol of RELINEEUROPE REE4000 – With support of the EBs, the real-time performance monitoring of the UV-emitter and the pre-selected performances are not only permanently checked and adjusted, but also shown and fully documented in the curing protocol.

RELINEEUROPE AG

Auftraggeber	Baustelle
46	75

Sanierungsangaben		Schlauchliner-Angaben	
Baustellennummer:	DN1700	**Liner-Hersteller:**	rh
Haltung:	57	**Liner-Nr:**	jg
Rohrdurchmesser (mm):	1700	**Herstellungsdatum:**	07.04.2016
Länge (m):	9,00	**Verbunddicke (mm):**	0,00
Aussentemperatur (°C):	10	**Lagertemperatur (°C):**	0
Wetter:	sonnig	**Lichtquelle (W):**	3x6000
Speicherzyklus (s):	30		

Kommentar:	

Bearbeiter:	dfg

Erstellungsdatum:	Montag, 9. Mai 2016 07:44:12

Datum & Zeit	Länge	Geschwindig.	Druck	Temperaturen				UV-Lampen										Assistent
Zeitstempel	(m)	(m/min)	(mbar)	T1 °C	T2 °C	T3 °C	T4 °C	1	2	3	4	5	6	7	8	9	10	Aktiv
21.04.2016 12:25:12	5,48	0,00	213.000	92	54	22	20	3.066	3.104	3.092	1.436	0	0	0	0	0	0	-
21.04.2016 12:25:42	5,48	0,00	210.000	93	59	24	20	3.064	3.101	3.095	E	0	0	0	0	0	0	-
21.04.2016 12:26:12	5,48	0,00	210.000	95	64	26	20	3.064	3.108	3.093	3.202	991	0	0	0	0	0	-
21.04.2016 12:26:42	5,48	0,00	210.000	96	70	28	20	3.066	3.100	3.091	3.111	1.022	0	0	0	0	0	-
21.04.2016 12:27:12	5,48	0,00	212.000	97	75	31	21	3.064	3.101	3.093	3.078	1.241	0	0	0	0	0	-
21.04.2016 12:27:42	5,48	0,00	213.000	97	80	33	21	3.061	3.104	3.090	3.073	1.750	0	0	0	0	0	-
21.04.2016 12:28:12	5,48	0,00	213.000	98	83	37	21	3.061	3.100	3.088	3.066	3.206	991	0	0	0	0	-
21.04.2016 12:28:43	5,48	0,00	215.000	99	87	41	21	3.061	3.103	3.087	3.067	3.120	1.038	0	0	0	0	-
21.04.2016 12:29:13	5,48	0,00	216.000	99	90	45	22	3.057	3.102	3.088	3.065	3.061	1.283	0	0	0	0	-
21.04.2016 12:29:43	5,48	0,00	217.000	99	93	48	22	3.056	3.101	3.085	3.066	3.062	E	0	0	0	0	-
21.04.2016 12:30:13	5,48	0,00	217.000	100	95	54	22	3.057	3.095	3.088	3.061	3.059	3.118	0	0	0	0	-
21.04.2016 12:30:43	5,48	0,00	218.000	100	97	60	22	3.057	3.098	3.084	3.065	3.059	3.039	0	0	0	0	-
21.04.2016 12:31:13	5,48	0,00	219.000	100	98	65	23	3.055	3.098	3.086	3.062	3.057	3.013	0	0	0	0	-
21.04.2016 12:31:43	5,48	0,00	220.000	101	99	69	23	3.053	3.095	3.085	3.065	3.057	3.013	0	0	0	0	-

Figure 8: Curing protocol of the UV-light curing system REE4000 from RELINEEUROPE

4 Integrated QR code scanner

With the QR code scanner integrated into the RELINEEUROPE curing system REE4000, the data of the hose liner can now be read directly from the goods receipt and all curing parameters are set automatically. The data for the required UV-light intensity and curing speed are stored in the control software. The required installation pressure in the hose liner is also displayed in the user interface. Errors due to incorrectly set parameters are thus reliably avoided.

Figure 9: QR-code scanner

5 Summary

A high-performance UV-light curing technology is the prerequisite for economical and high-quality pipe rehabilitation with UV light-curing GRP hose liners. The performance of the UV-light sources has so far been decisive for the curing speed and thus the cost-effectiveness of the rehabilitation job sites. For large diameters and liner thicknesses above 8 mm, the measurement of the resin reaction temperature on the liner surface by IR sensors can no longer be used as a previous hardness control.

In addition, more and more UV-emitters with different performances are being used. Performance control in real-time on site is usually not feasible with a conventional UV-light curing system.

Modern technologies in electronics (Electronic Ballasts – EB) now ensure such important power monitoring in real-time, so that especially large diameters up to DN 1900 with big liner thicknesses of over 10mm can be traceable cured with pure UV light. Real-time monitoring and documentation during the whole curing process make this technology comprehensible.

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Session 2

PRESSURE PIPE REHABILITATION

Status quo of the CIPP product standards for water and gas networks

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1. INTRODUCTION

Cured in place pipes (CIPP) gain increasing importance as a trenchless renovations technique. With a starting point in sewerage networks, new application fields are pressure sewers, water and potentially gas networks.
This is reflected by the development in standardization. In 2018 already the second edition of the underlying EN ISO 11296-4 was published: "Plastics piping systems for renovation of underground non-pressure drainage and sewerage networks - Part 4: Lining with cured-in-place pipes". The second edition of the Mother of CIPP standards appeared after the usual period for revisions, i.e. after about 5 years. As a result of the growing experience with the rehabilitation process, there were a number of clarifications and extensions.

The new edition of EN ISO 11296-4 was accompanied by issuing the new EN ISO 11297-4, which extends the application field for CIPP to sewerage networks under pressure. Within the product family, EN ISO 11298-4 is the product standard for water networks. A DIS document (draft international standard) is already published and publicly available. The final step for the standardization work will be the document EN ISO 11299-4, which will cover CIPP requirements to be applied in gas networks. The project is currently on hold, i.e. recently deferred by ISO/TC138/SC8/WG4 as CIPP for gas is assessed as not yet state of the art.

This paper describes the status quo of standardisation for the different renovation techniques with regard to the planned applications. It discusses the similarities and differences between the application-specific standards and classifies the state of development.

2. CIPP STANDARDS AS MODULAR SYSTEMS

The standards for CIPP within the product family can be seen as a modular system. The requirements are specific regarding the intended use and partly, they are getting more sophisticated step by step (Figure 1).

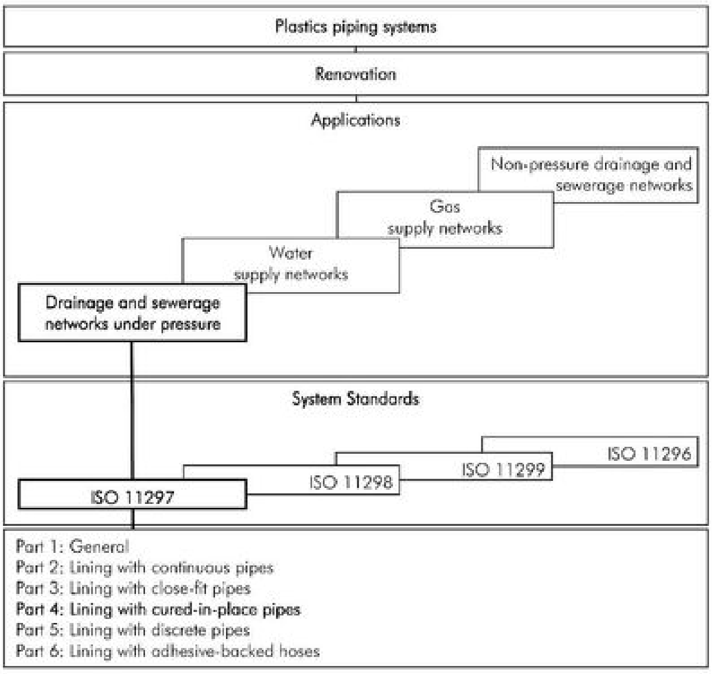

Figure 1. CIPP product and standardization family, diagram from EN ISO 11297-4

EN ISO 11297-4 adds the pressure requirements to liners in sewerage networks covered by EN ISO 11296-4. This is first of all the demand for a significantly higher circumferential strength of the CIPP products. That is why, glass reinforced CIPP have a much higher relevance for this application.

With requirement for long term strength against pressure established in EN ISO 11297-4, the product standard for CIPP in water networks, EN ISO 11298-4, sees new challenges in the field of hygienic requirements. Basically, there are also new challenges in the area of lateral connections because pure transport lines are rare and regular lateral connections are the rule. The treatment of this topic in the standard is discussed in point 5. At the same time, there are lower requirements in other fields for water networks: temperature and cyclic loads. In general, the diameters are also smaller.

Developing CIPP products for gas networks has again similarities with the liners in sewerage pressure networks. Anyway, there will be special requirements in future EN ISO 11299-4, for example in the field of gas permeability or chemical resistance. Also, the safety level in design is supposed to be higher, since a failure of CIPP is connected with much more potential damage for the environment.

3. COMPARING EN ISO 11297-4 WITH EN ISO 11298-4 – SCOPE, STRUCTURE AND MECHANICAL CHARACTERISTICS

EN ISO 11297-4 has numerous references to EN ISO 11296-4 and can be seen as a relatively lean standard addition for the first one, which specifies requirements and test methods for CIPP in sewerage networks resulting from internal pressure. At the same time, EN ISO 11297-4 sets in its scope clear boundaries regarding temperatures up to 50 °C and states, that the standard does not include requirements or test methods for resistance to abrasion, cyclic loading or impact.

EN ISO 11298-4 is a more comprehensive and self-standing document with less references. Despite of the typical forms of rather branched water networks without manholes at bends, according to the scope the pressure rating of CIPP liners passing through bends, is not covered by this standard. This reflects the current state of the art in technical development.

In general, however, the structure of EN ISO 11298-4 is almost identical to that in EN ISO 11297-4. The chapters on mechanical and physical properties have been almost completely adopted. This is hardly surprising because the mechanical action from internal pressure for the liner from wastewater and water are the same. Of course, there is no strain corrosion test in EN ISO 11298-4 as in the standard for sewerage systems. Only the requirement for the ultimate elongation is less challenging with 0.5 per cent, which remains slightly erratic (Figure 2).

Characteristic	Requirement[a]	Test parameters		Test method
		Parameter	Value	
Initial circumferential tensile wall strength	Declared value in N/mm, i.e. ultimate circumferential tensile force per unit length of pipe	In accordance with ISO 10639:2017, 5.3.6, limited to method A		ISO 8521:2020, Method A
Initial specific ring stiffness, S_0	Declared value, but not less than 750 N/m^2 [b]	Number of test pieces	2	ISO 7685:2019[c], Method A or Method B
		Test piece length for:		
		— $d_n \le 300$ mm	$d_n \pm 5$ %	
		— $d_n > 300$ mm	(300 ± 15) mm	
		For method B: relative deflection	(3 ± 0,5) %	
Short-term flexural modulus, E_0	Declared value in MPa	Number of test pieces	5	Annex B
		Speed of testing	10 mm/min	
Flexural stress at first break, σ_{fb}	Declared value in MPa	Sample orientation	Shall conform to 8.8	
Flexural strain at first break, ε_{fb}	Declared value but not less than 0,75 %			
Ultimate longitudinal tensile stress, σ_L	Declared value in MPa	Number of test pieces	5	ISO 8513:2016[c], Method A or Method B
Ultimate elongation	Declared value but not less than 0,5 %	Speed of testing	5 mm/min	

[a] Declared values refer to the mean value of each characteristic as determined from the results of tests on a set of the specified number of test pieces.

[b] The minimum required value may be reduced where the surge pressure is higher than 20 kPa absolute (−80 kPa gauge pressure). The absolute minimum ring stiffness shall be 250 N/m^2.

[c] In case of dispute, Method A applies.

Figure 2. Table 4 of EN ISO 11298-4 "Short-term mechanical characteristics of pipe samples"

It is somewhat surprising that subclause 5.1, Materials, in ISO/DIS 11298-4 does not differ from that in EN ISO 11297-4. (The latter one refers simply to EN ISO 11296-4.) Table 1 allows the same materials for CIPP in water networks as in wastewater networks (Figure 3). In particular, there are no restrictions on the resin system used and the thermoplastics for the permanent internal membrane, and no general requirements for an internal membrane in combination with certain resin systems.

Lining tube component	Materials
Resin system:	UP, VE or EP [a]
— resin type	None, inorganic or organic
— filler type	Heat-initiated, light-initiated or ambient cure
— curing agent type	
Carrier material/reinforcement	Polymeric fibres: PA, PAN, PEN, PET, PP or PPTA
	Glass fibres of types 'E', 'C', 'R' and/or 'E-CR' conforming to ISO 10639
	Carbon fibres of declared designation conforming to ISO 13002
	Combinations of the above fibres[b]
Membranes	Unrestricted[c]

[a] Other resin systems are current state of the art and although excluded from the scope can in principle be tested in accordance with this standard.

[b] Where a combination of fibres is used, the proportions by mass of each fibre type shall be declared to within 5 %.

[c] As there are no requirements for membranes, there are also no restrictions on the choice of thermoplastic materials used for membranes.

Figure 3. Table 1 of EN ISO 11298-4 "Materials for lining tube components"

That is astonishing, since liner manufacturer with national approvals for renovating water networks, clearly distinguish in resin systems for different applications. German manufacturer SAERTEX multiCom for example uses UP resin systems only for applications in sewerage. For applications in water networks where the SAERTEX-LINER® H2O has national approvals in various countries, only a styrene free VE resin system is used.

However (and this is most likely the explanation for the high degree of freedom discussed above), in subclause 5.3, Material characteristics, where only the glass transition temperature for the resin systems is regulated in the standards for sewerage networks, there is a further requirement regarding the properties with an impact on the hygienic requirements:

"Regarding the effect on water quality, in addition to the requirements of ISO 11298-1:2018, 5.3, attention is drawn to the requirements of national regulations."

Obviously, it was not feasible at ISO level to agree on more uniform regulations here, because the requirements for the quality of water as an essential nutrition item are very different worldwide. Also ISO has to defer to national regulations and cannot override them.

4. CIPP FOR WATER NETWORKS – MEASURING EFFECTS ON WATER QUALITY

In Germany, for example, the hygienic assessment of the effects of CIPP materials on water quality is carried out on the basis of the guideline of the Federal Environment Agency. According to this guideline, tests are conducted in which liner samples are immersed in water for defined periods of time and the changes in certain water characteristics are measured regularly.

The German guideline requires the measurement of the following characteristics of the water:

- Colouration
- Turbidity
- Tendency for foaming
- Smell threshold value
- Taste threshold value
- Total organic carbon content (TOC).

The TOC is measured according to EN 1484 with a defined surface-to-volume ratio of 5 dm^{-1}. The data set of the measurement for the SAERTEX-LINER® H2O is shown below in Figure 4. The limit value after 31 days, which is 0.5 mg/l is clearly complied with.

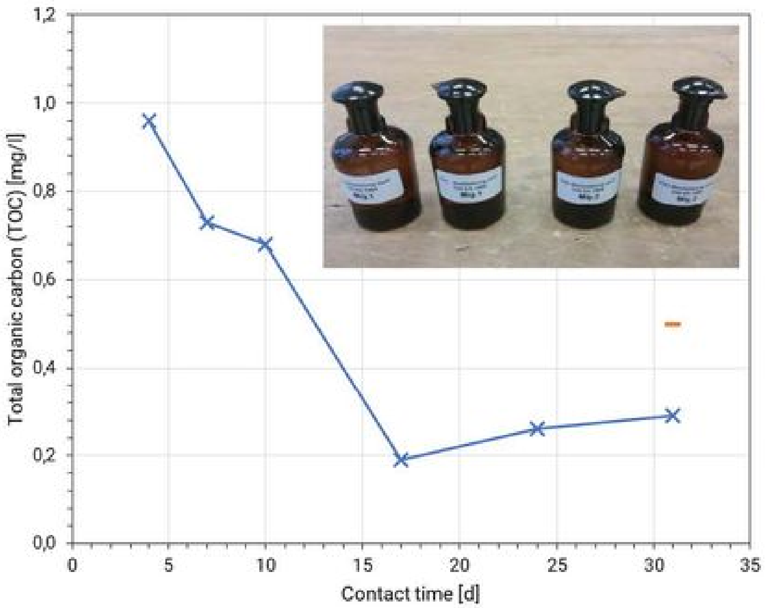

Figure 4. Data set of SAERTEX-LINER® H2O, measurement of TOC in contact water with time

5. LINER TERMINATIONS AND LATERAL CONNECTIONS

In general, the connection of the CIPP to the existing pressure networks such as sewerage, water or gas is technically demanding. For wastewater applications, there are various solutions for liner terminations that in principle can also be transferred to the water sector. The Amex Liner End Seal of the SAERTEX-LINER® H2O is displayed in Figure 5.

Figure 5. Amex Liner End Seal of the SAERTEX-LINER® H2O

The EN ISO 11298-4 regulates in subclause 8.7.1 under "Additional characteristics" of CIPP in "I" stage the test requirements for the tightness of the connections (Figure 6).
These requirements with 150 per cent of MDP and a negative pressure of 0.8 bar are identical to those in EN ISO 11297-4 and typical for pipes in pressure networks. Although this requirement is in line with EN 805, the requirement regarding negative pressure is comparatively strict for water networks because pressure surges with negative pressure hardly ever occur here. This is a phenomenon that is more familiar from sewerage pressure pipes.

Characteristic	Requirement	Test parameter		Test method
		Parameter	Value	
Initial leak tightness	No leakage	Test pressure	1,5 × PN	ISO 7432 or ISO 8533 as applicable
		Time	15 min	
External pressure differential	No leakage	Test pressure	−80 kPa	ISO 7432 or ISO 8533 as applicable
		Time	1 h	

Figure 6. Table 6 of EN ISO 11298-4 "Leak tightness of liner terminations"

It is interesting to note that the requirements regarding the material properties affecting the hygienic requirements (see Chapter 3) do not apply to the liner terminations. Even though the liner terminations are of course only short sections within the sections to be renovated. For connections, elastomeric materials are usually used, which are known to diffuse organic substances, for example.

A technically much more demanding issue than the liner terminations are the laterals. Here, there is usually a change in nominal size. EN ISO 11296-4 has regulated this topic for unpressurised applications through various requirements. However, in this field the technical solution is much simpler.

With regard to laterals, there are currently no requirements in EN ISO 11297-4 and EN ISO 11298-4, which means that the scope of application is not formally, but in practice limited to transport pipelines. For sewerage networks, this is certainly a restriction with less major effects. The transport lines upstream close to the treatment plants are usually the network components that are operated with pressure and rarely have laterals.

For water networks, the restriction is more relevant. The greatest need for rehabilitation is in the inner-city areas, because that is where the networks are oldest. Here, however, there are hardly any sections without laterals. In this respect, there is a great need for technical development in this field. Currently, the most common research approach is to use metallic parts with sealing elements, as is the case with SAERTEX multiCom (Figure 7).

Figure 7. Example of product development for laterals, SAERTEX multiCom

The solutions currently being pursued are not intended for trenchless installation. The second step to the next level will be trenchless solutions. It is expected that it will take a few more years for this type of development.

7. NATIONAL CERTIFYING PROGRAMS FOR GAS APPLICATIONS

Since the product standard EN ISO 11299-4 is still under development and many countries with a demand for trenchless renovation techniques in gas networks see a significant potential of CIPP products, various national certifying programs were launched.

One example is the program conducted by the independent test center WRc. The goal is to create a performance specification proving liner systems are 'fit for purpose' as a rehabilitation technique for iron gas distribution mains up to 2 bar pressure in Great Britain.

Apart from general testing of mechanical characteristics of the CIPP as in aforementioned EN ISO 11297-4, in the program tests were tailored for the intended application. Gas permeability testing was naturally considered a key parameter. The test gas used was a 50/50 mix of hydrogen (H_2) and methane (CH_4) at 2 bar pressure and ambient temperature. The test gas was in contact with the internal surface of the CIPP liner material and on the external liner surface a continuous flow of N_2 gas transported any permeating gas to chromatography detectors. The gas permeability of SAERTEX-LINER® GAS was found to be adequate for the intended use.

Another specific requirement was impact testing with a 5 kg impact tup mass that was dropped from a 2 m height on the crown of the liner. Subsequently, the liner was in a 3 bar pressure test for 24 hours. In the following burst pressure test, regular values were achieved and no failing on the impact point was observed for the SAERTEX-LINER® GAS.

Strain corrosion testing with other chemicals as described in EN ISO 11297-4 was conducted, to demonstrate if any of the chemicals are detrimental to CIPP performance. The key parameter is tensile strength, which after 6 months immersion must not decrease by more than 20% of the mean control (non-immersed) samples value. SAERTEX-LINER® GAS again passed the test.

Eventually, testing was conducted in a buried pit, to investigate CIPP liner integrity during catastrophic failure of a cast iron gas host pipe. Nominal 12" diameter cast iron pipe had been sourced from an abandoned main and buried and lined with a SAERTEX-LINER® GAS. The CIPP liner was pressurized to 2 bar pressure with nitrogen and then hydraulic beams used to apply loading to the surface. The loading was increased until the gas main fractured (Figure 8). The SAERTEX-LINER® GAS subsequently proved its integrity by its ability to continue to retain internal pressure.

Figure 8. Fractured gas host pipe after loading in pit test and integer SAERTEX-LINER® GAS

8. REFERENCES

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KEY DESIGN CONSIDERATIONS FOR PE 100 AND PE 80 PRESSURE PIPE LINERS

Steven Brogden, Dr.

Well established methodologies exist for designing stand-alone pressure pipes using PE 100 and PE 80 materials, embodied in system standards such as EN 12201 and EN 1555 for water and gas pipes. Conservative liner designs can utilise these standards with high safety factors, providing that this is both economically viable and that reduction in flow capacity is not prohibitive.

Aged iron, ductile iron, steel, GRP and PVC pipes often need rehabilitating when in largely good condition other than leaks at joints and/or small pinholes through the pipe body. In these cases, the host pipe will invariably be structurally sound and expected to retain its hoop strength for many years.

Key design properties for the PE liner become gap bridging and hole spanning, for which methodologies were developed over 20 years ago, rather than hydrostatic strength. Using such an approach can lead to the design of very thin liners down to SDR 51 for example, where an SDR 17 may have been originally considered for a 10-bar water pipe. For gas and other hydrocarbon applications, diffusion through the permeable polyethylene contributing to the vacuum collapse condition prohibits use of thinner liners.

Close fit liners can be tailor made for the application.

1. INTRODUCTION

Many cast/ductile iron, steel, cementitious, PVC and GRP pipelines lose integrity or function resulting in fluid loss, low flow or poor water quality. A variety of reasons include corrosion, tuberculation as in figure 1, joint leakage and low toughness. A common scenario especially for the many aged cast iron pipes in the water system, is a combination of leaking joints and/or small pinholes in the pipe body. In many of these cases the pipe often still demonstrates high ring stiffness that could support itself and a liner for many years in the future.

The decision of the asset owner is whether to decommission, replace or rehabilitate. Even if it is only the joints leaking, repairing each joint will rise to a cost comparable with open cut. Replacement is very costly and especially so where new towns and cities have grown up over and around the pipeline, perhaps since Victorian times. In the big cities of northern Europe, frequently such pipelines have been left with ongoing leakage issues as it has been considered too expensive and/or disruptive to replace them.

A further issue is that few rehabilitation techniques are suitable in such an environment where above ground space is limited and the presence of bends/other services makes any intervention difficult and costly. Budgets are limited so a water utility must consider leakage, water quality, hydraulics & structural condition before deciding where to spend precious replacement or rehabilitation costs & resources. With water supply often running close to capacity, it is frequently essential to maximise flow both for now and future proofing of the systems. The full advantages of polyethylene lining and especially tight fit lining maximising flow can only be achieved by a versatile design process enabling selection of the optimum liner thickness whether that be SDR 11 or preferentially a thinner liner tending towards the goal of SDR 51.

Figure 1. Small diameter cast iron water pipe showing reduced flow capacity due to tuberculation.

2. DESIGN OF PE 100 AND PE 80 PRESSURE PIPES AND APPLICABILITY FOR LINING

High and medium density polyethylene materials have a long and successful track record in low pressure water, sewerage and gas pipelines. This is especially over the last 40 years where the materials evolution has progressed from PE 80 and then PE 100 to PE 100 RC. The ultra-high ductility, flexibility and inertness of these materials enables them to deliver fluids without corrosion, maintaining a smooth pipeline bore throughout long lifetimes. Failure rates of the pipelines themselves are more or less negligible and as long as welds and mechanical fittings are undertaken correctly there is little or no risk to the integrity of these systems. Modern PE 100 RC materials display the following attributes for transporting potable water:

- Material with water quality approval and compliance with EN 12201-1
- Chemical resistance under operating conditions
- Superior weldability in pipe form
- Low sag characteristics in pipe form
- Superior slow crack growth resistance in pipe form
- Long term hydrostatic strength in pipe form
- Rapid crack propagation resistance in pipe form
- PE 100+ Association approval
- Long successful track record with reduction and folding technologies
- Available from many pipe manufacturers to EN 12201-2

The downside is that the beneficial short and long-term visco-elastic behaviours of thermoplastic pipes, which provide these wonderful properties, are complex. Mechanisms such as stress relaxation and creep with their different elastic and in-elastic stress and strain components are all affected by small changes in strain rate and temperature. These relationships and their effects are a design challenge, often not well understood and sometimes mis-interpreted by specialist pipeline design engineers more comfortable with rigid metallic and cementitious materials.

For example, PE 100 RC pipes are qualified according to the water pipe system standard EN 12201-2 based upon the long-term creep strength according to ISO 9080. What is most important for these highly crack resistant stand-alone pressure pipe grades is that the three key design conditions of the PE 100+ Association are met:

1. Long term creep strength according to ISO 9080
2. Long term stress crack resistance according to EN ISO 13479
3. Short term resistance to rapid crack propagation according to EN ISO 13477/8

2.1 Long term creep strength according to ISO 9080

The foundation of EN 12201 is that the PE 100 material is classified according to EN ISO 12162 because it has a long-term hydrostatic strength of a minimum of 10MPa for 50 or 100 years. Pipe material suppliers such as Borealis are required to test a series of pipes under pressure at different temperatures and times in order to produce "design curves" based upon the long-term creep behaviour of the pipes according to ISO 9080. Use of the well-established Arrhenius relationship enables extrapolation to a 100 years lifetime by testing for more than a year at 80°C. An example is Figure 2, showing the regression data developed in Element test report P-09/38-v2 commissioned by Borealis.

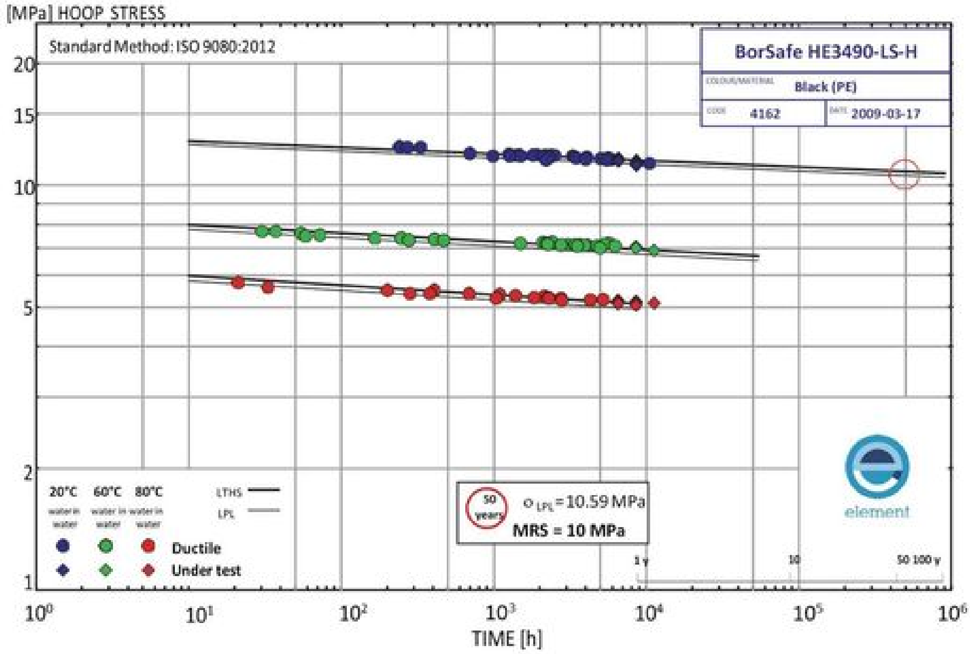

Figure 2. 50-year extrapolation in Element test report P-09/38-v2.

The hoop "strength" of HE 3490 LS-H pipes is given here as a lower prediction limit of 10.59 MPa at 50 years. Testing of long-term pipes has enabled extrapolation to 100 years with a lower prediction limit of 10.47 MPa as a 100-year design basis. What this means is that a pipe designed to its pressure rating would need to be loaded continuously at its safety factor of 1.25 times that pressure underground at 20°C for it to burst after 100 years. This is an extremely unlikely scenario as in practice pipes are usually operated at significantly below their pressure rating, not always continuously and at average annual temperatures for buried pipes below 20°C. The mechanism of this mode of failure is creep, which is the` extension or strain placed on pipes due to continuously applied load, internal pressure in this case, over long periods of time. It doesn't mean that 100 years is the limit for pipe lifetime. For short term loading events the pipe will still be behaving like a new pipe as there will have been insufficient time for oxidative degradation processes to reduce the original physical properties of the material.

For a tight fit liner, there is little or no creep as the outside of the PE 100 liner pipe is constrained by the host pipe. Apart from creep into small spaces, gaps and holes which require separate consideration, the result is that this failure mode is impossible and does not require designing for. Thus a 100 year plus lifetime plus is demonstrated for a PE 100 liner for the major design criterion of plastic pipes. Reasonable proof that this design condition is irrelevant for a constrained liner is demonstrated by many oil-well water-injection lines operating for decades with thin wall tight fitting PE 80, PE 100 and PE 100 RC liners at hundreds of bar internal pressure.

2.2 Long term stress crack resistance according to EN ISO 13479

The second mode of failure important for consideration when designing thermoplastic pipes is slow crack growth or stress crack resistance according to EN ISO 13479. This test came about after early experiences with 1st generation HDPE pipes and PVCu pipes made from the 1950s onwards, where the brittle failure mechanism of slow crack growth was observed in many utility pipeline failures.

From the mid-1980s the introduction of bi-modal PE 80, followed by PE 100 and most recently PE 100 RC grades has almost entirely eliminated slow crack growth as a mode of failure, apart from under very exceptional circumstances. The mechanism is associated with being close to the melting point of the material resulting in fracture and eventual failure in shorter times as any test temperature is increased. It occurs due to the gradual growth of cracks from sharp defects combined with point loading, until there is insufficient material left to withstand operating stresses and subsequent strains at which point final catastrophic failure occurs. Thus, testing in accordance with EN ISO 13479 is conducted on notched pipe samples at 80°C, in order to accelerate fracture to manageable durations.

The continued focus on this failure mechanism, and various new tests devised by many testing institutes together with some of those promoting bimodal PE pipe grades, does the material a disservice. It can be falsely perceived as a weakness of the material, whereas it is a strength of the material wherever its plastic properties are an advantage, gladly seized upon by those such as the ductile iron pipes lobby.

PE 100 materials which meet the PE 100+ Association requirement of 1000 hours will be extremely unlikely to have any problems from slow crack growth within 100 years, if correctly designed and installed. It is worth noting that this is a fracture mechanics mechanism that is favoured by the plane strain condition found in thick-walled pipes, rather than the plane stress condition found in thinner pipes or liners. A reduction in safety factor not only means that material is being saved, and that fluid flow is being increased, also that stress cracking is even less likely.

As for the mechanism of creep rupture, slow crack growth is also associated with significant strain on a pipe, in this case to propagate a crack subjected to stress. A tight- fitting liner is effectively constrained within and against the internal bore of the host pipe thus making this mechanism certainly implausible if not impossible. Very large strains could only be introduced on the outside of the liner if some internal defect were to protrude a very significant distance into the host bore. All high-risk defects are removed prior to lining or it would be impossible to install the liner.

3.3 Short term resistance to rapid crack propagation according to EN ISO 13477/8
The third key design criterion for PE 100 pipes is resistance rapid crack propagation, tested according to EN ISO 13477 or EN ISO 13478. As the name suggests this is a fast fracture mechanism and although a material qualified by the PE 100+ Association such as HE 3490 LS-H exceeds resistance of more than 10 bar at 0°C, it is difficult to envisage how this can be relevant to a thin-walled liner constrained within a rigid host pipe with flowing water. Again, the plane strain condition of a thick-walled pipe is required as a medium for the crack to initiate. The event is only applicable for gas pipes as a significant build-up of gas within the pipeline is required combined with a large external driving force such as the bucket of an excavator suddenly puncturing the pipeline.

The three key design parameters for PE 80/PE 100/PE 100 RC pressure pipes are associated with long term creep strain and/or thick-walled pipelines. These parameters are of much less importance when considering design of a tight-fitting thin-walled liner. A thin-walled liner largely subject to stress relaxation rather than creep requires different design criteria.

3. STRUCTURAL CLASSIFICATION OF PRESSURE PIPE LINERS

A brief description of the lining techniques required to introduce a PE liner into a host pipe for rehabilitation is necessary. For this paper the different types of liner have been broken down into 4 categories;

a) Slip lining – where a fully-structural full pressure bearing liner is installed by pulling and/or pushing the liner pipe into a host pipe. Although in common usage, the asset owner often must accept a significant loss of hydraulic capacity.
b) Die drawing – where a liner of any thickness or size (SDR 11-51, 100mm to 1500mm) is pulled through a bespoke reduction die in the insertion pit by means of a winch at the reception pit. Has the potential to offer the optimum liner thickness improving hydraulic capacity. Rarely used due to absence of appropriate design/equipment knowledge and often needs bespoke pipe sizes. See figure 3.
c) Roll down – where a limited range of smaller thicker pipes (~SDR 11-33, <500mm diameter) are pulled through a set of reducing rollers at the insertion pit with a winch at the reception pit. Rarely used to limited size range, need for specialist equipment and often needs bespoke pipe sizes.
d) Folded liners – where a bespoke liner is produced and then folded before being pushed and/or pulled into the host, requiring post installation pressurization making it difficult to help achieve a concentric close fit. Limited by size ranges that are either small diameter fully-structural or large diameter semi-structural.

Figure 3. Die drawing a 1030mm SDR 51 pipe on the Vyrnwy aqueduct, United Utilities.

PE 80 and PE 100 liners for rehabilitation of drainage, sewerage, water, and gas pressure pipes are classified in EN ISO 11295. An independent pressure pipe liner, is on its own capable of resisting all applicable internal loads without failure throughout its design life. Whereas an interactive pressure pipe liner relies on the existing pipeline for radial support in order to resist without failure all applicable internal loads throughout its design life.

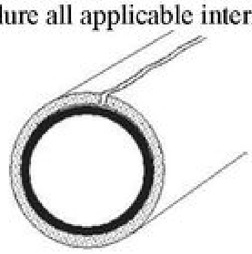

Figure 4. Class A, independent fully structural liner.

According to the classification system EN ISO 11295 such liners are either Class A, independent fully structural liners see Figure 4 or Class B interactive semi-structural liners see Figure 5. Class A liners can either be loose fitting, as a slip-liners or close fitting as in die drawn or rolldown pipe. Die drawing or rolldown can also be used for loose fitting liners, though this is not gaining the maximum benefits of these reduction technologies. Class B interactive liners cannot be installed by slip lining and must use die drawing or rolldown. Folded liners are another option that would normally be used for interactive close fit lining, though in some rare circumstances they could be used as Class A fully structural liners for pipelines operating at very low pressure.

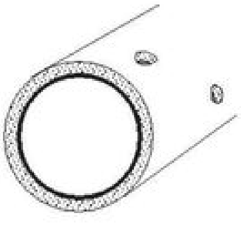

Figure 5. Class B, interactive semi-structural liner.

Design considerations for a fully structural liner are straightforward as we can consider the liner in the same way as a full pressure bearing pipe. Although there are some additional risks such as thermal strain when a slip liner is "locked-off" by the end fittings on the host pipe and those associated with eventual complete failure of the host, design can be undertaken with negligible risk using the often over-design principles requested by many utilities. It may be the policy of a water company to specify say SDR 11 for all its pipes, even those operating at a few bar. This may be considered an unnecessary waste of resources for stand-alone open-cut pipes as well as making the choice of thermoplastic pipe uncompetitive to traditional materials. When such a design principle is applied to a fully structural liner the policy becomes even more dubious, as even with the advantages of die drawing or rolldown helping to increase the bore, flow capacity may be reduced from that of the original pipeline.

Maintaining flow rate is usually the most critical factor when rehabilitating with a liner. An aim of this paper is to demonstrate that by using thinner interactive close-fitting liners, flow rate can be increased from that of the original pipeline. This is achieved with a much lower cost solution not only in terms of pipe material but also ease of installation.

Definitions of close-fitting vary throughout industry specifications from being 90-95% of the host diameter to an absolute tight fit everywhere within the host. No PE thin wall lining system can demonstrate a truly tight fit, though some systems are better at achieving this than others. Both rolldown which is limited to relatively thick pipes, and various folded liner systems limited to relatively thin liners, require post pressurization to achieve a close fit, potentially leaving substantial gaps in areas of the host pipe. Such gaps will increase upon depressurization. Die drawing with a project specific design and pipe size imparts more elastic diametric strain than axial strain with a tendency to tighten onto the host in most locations due to continued radial expansion post termination. No method is perfect although as long as it delivers a liner that is close enough to the host wall for the additional strain imposed by internal pressure to reach the sides of the host it will be able to act as an interactive liner with support from the host.

Much of the remainder of this paper will focus on the interactive close-fit lining solution as it is an area that pipeline owners have often shied away from due to uncertainty of how to define and understand the different key design criteria. The criticality of much of our aged pipeline network demands that we as an industry try harder to solve this.

4. DESIGN OF THIN WALL LINERS USING PE 100 AND PE 80 MATERIALS

PE 100 pipes are used extensively as stand-alone pressure pipes. When used for lining they are frequently used as fully structural liners. There is a much greater potential to use these materials as semi-structural or interactive liners where the aged pipes may just be leaking at joints or through pinholes. Whilst it is almost always beneficial to have a tight-fitting liner, it is essential that a tight-fitting liner process is used when designing a semi-structural, interactive liner. In these cases, the liner is relying on the host pipe to provide structural restraint removing the hoop stress design element and requirement for high ring stiffness.

It has been established in Section 2 that none of the three major failure conditions are relevant for a thin-walled tight-fitting PE 100 liner, constrained within a structurally sound host pipe, transporting potable water. The question is what are the appropriate design considerations for a thin-walled interactive semi-structural liner?

The only locations in the host pipe where the liner is at risk from creep mechanisms are those smaller areas where there could be a gap at a joint or there are small pinholes in the pipeline. Fortunately, bimodal polyethylene pipes have a perhaps surprisingly high capability to span over such spaces over a long lifetime. A comprehensive research program undertaken by Bradford University in the 1990s determined the capability of polyethylene pipes to both bridge gaps and span holes or voids. This work on PE 80 can be used to determine the behaviour of PE 100 materials too as long as specialist polymer engineering principles are carefully followed and conservative assumptions made. Using an appropriately high safety factor is recommended (eg. 2), thus enabling the work to act as a design guide. When being designed as a semi-structural liner it has to be presumed that lifetime is also dependent on the host pipe maintaining ring stiffness for external loadings and that gaps or holes will only open to a certain extent within the future design life.

Another possible failure mechanism that could occur is due to the liner having insufficient collapse resistance or ring stiffness to withstand an internal vacuum condition from a pressure transient and/or the presence of groundwater. While other internal loadings such as the pressure test for commissioning the pipeline and thermal effects especially on end fittings need considering, most other external loadings do not need to be considered if the host pipeline is and will continue to be structurally sound. Therefore, the key design criteria for an interactive thin wall liner are:

1. Gap bridging capability
2. Void spanning capability
3. Collapse resistance

4.1 Gap bridging capability

Figure 6. Gap in spigot and socket joint.

For many thin wall lining projects, the gap bridging capability of the liner over a joint is the most critical design parameter, such as in Figure 6. Boot et al undertook extensive research in the 1990s at Bradford University on the structural performance of thin-walled polyethylene pipe linings for the renovation of water mains. Physical testing was combined with sophisticated finite element analyses using ABACUS software to model behaviour of PE 80 liners bridging gaps and spanning holes or voids. Complex modelling of viscoelastic behaviour of the thermoplastic material was used in these analyses to provide 50-year predictions of the resistance of gaps and voids of liner material to bridge and span under pressure. By using very conservative assumptions, not least that maximum operating pressure is applied throughout lifetime, design curves can be created appropriate for the PE 100 materials in use today. The author is aware that that others have also made this conclusion. From the design curves presented by Boot et al a conservative estimate of gap bridging capability can be made, in line with an agreed estimate with pipeline design engineers for the remaining life of the pipeline being lined.

4.2 Void/hole spanning capability

Using similar methodology and conservative assumptions a void or hole spanning capability can also be calculated from the work of Boot et al at Bradford University.

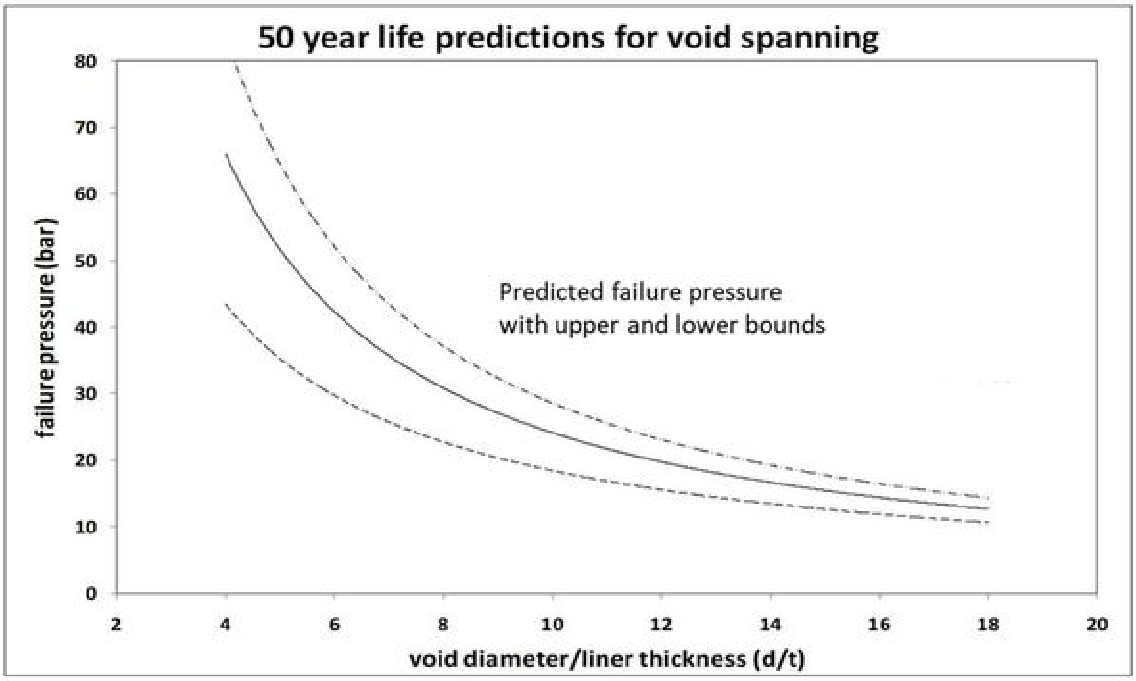

Figure 7. 50-year life void/hole spanning predictions for a PE 80 lining.

The relationship in Figure 7 is taken directly from the referenced paper. In this case a series of tests were used to confirm the applicability of a circular hole or void to represent the area for other uniform geometries.

4.3 Collapse resistance

For a liner to collapse internally a common mechanism is through diffusion of gas molecules into any micro-annular space between the liner and the host pipe combined with a sudden depressurisation, especially at high temperature. Fortunately, this mechanism is of little concern for a low-pressure potable water pipeline, operating at ambient temperature, where any gas dissolved in the water is negligible. Whilst this paper is focussing on water pipelines, and though tight fit polyethylene lining technology is suitable for drainage, sewerage, industrial and low-pressure gas, it is not suitable for use with high pressure gases. It is this collapse mechanism that restricts the technology to applications where no more than few bar of dissolved gas can be tolerated in the transported fluids. This is why the technology is ideally suitable for use for high pressure water injection but not high-pressure hydrocarbon applications, unless the annulus can be vented in some way.

A possible risk in the low-pressure water applications being considered here is the ingress of ground water from a very high-water table during a subsequent depressurisation event being sufficient to cause liner collapse. The impact of surge and fatigue events should also be assessed in conjunction with this. Guidance can be found in the UK WIS IGN 4-37-02. Summarising, this guidance demonstrates that a surge event of twice the magnitude of the pressure pipe rating can be withstood by a PE 100 pipe and that fatigue is not a relevant failure mechanism for PE compared to PVC. Since it is the increase in stiffness of the PE 100 pipe that imparts the resistance to rapid surge pressure, it is considered that a PE 100 pipe acting as a constrained liner within a rigid host will have at least as much, if not greater, resistance to surge events. This will also be true for the collapse condition as higher modulus increases collapse resistance. Fatigue is a similar fracture mechanism to slow crack growth and as described earlier there is insufficient strain to either initiate or propagate this type of crack within a thin constrained liner.

From the simple relationship determined by Glock, there have been many attempts to model the behaviour of elastic liners enclosed within a rigid pipe. The many different assumptions, both of the form of the formula and the material property inputs that have been used, present a very wide range of solutions for close-fitting plastic liners enclosed within a steel pipe. Very little physical testing has been undertaken on true tight-fitting polymer liners over a range of controlled temperature conditions. Occasionally misinterpretation has led to catastrophic failures for polymeric lined pipes, where dissolved gas has been present in the transported fluids, usually at high operating temperature. In some of these cases, using much lower conservatism than Glock, attempts to gain benefit from the additional support provided by the host pipe have been incorporated well beyond what the theory suggests. Other less conservative models including the behaviour of defects within the model do not account for practical test experience and appropriate inputs for short-term loadings on polyethylene liners.

A true appreciation of the fundamental properties of polymeric materials is required in order to develop appropriate formulae across a range of temperature and operating conditions for these materials. For these reasons the liner collapse model considered most appropriate is in very close agreement with the original Glock relationship, albeit with corrections for polymer constraint and the assessment of defects within the host pipe. Although a safety factor of 1.33 is often recommended in Oil & Gas specifications, an appropriate safety factor of 1.5 is recommended here. This allows the selection of an SDR rating for a liner in a host pipe where wider tolerances of host pipe diameter can be accommodated with reasonable conservatism.

The conditions for a collapse event of the types considered above are always dictated by the short-term modulus of the polyethylene material rather than the long term, hence it is appropriate to use a short-term elastic modulus in the equation. Boot developed equation 1 below:

$$p_{crit} = 10.k.\left(\frac{E}{1-\nu^2}\right).\left(\frac{D}{t}\right)^m \qquad [1]$$

p_{crit}	critical collapse pressure (bar)
k	coefficient depending on imperfection size (one lobe solution)
E	short term elastic modulus (MPa)
υ	Poisson's ratio
D	average diameter of the liner once installed (m)
t	liner wall thickness (m)
m	power depending on imperfection size (one lobe solution)

5. COMPARISON OF LINER DESIGNS USED FOR DIFFERENT DIE DRAW PROJECTS

The key performance indicator for a pipeline owner is invariably hydraulic capacity. In order to demonstrate the relative effectiveness of the different lining projects below, they have been compared using a hypothetical flow rate of $0.2m^3/sec$ using the Hazen Williams equation.

All the examples in Table 1 below are current or recently completed Projects by Die Draw Ltd in Northern Europe, providing solutions across a wide SDR range dependent on the individual circumstances:

Pipeline	A	B	C
Host material	Cast iron	Cement lined ductile iron	Steel
Host ID	914.4mm	399.2mm	997.5mm
Design pressure	6 bar	8 bar	10 bar
Minimum ring stiffness	10kPa	n/a	n/a
Installation length	500m	365m	600m
Liner material	PE 100	PE 100 RC	PE 100 RC
Liner type	Fully structural	Semi-structural	Semi-structural
Initial liner OD	900mm	425mm	1045mm
Liner SDR	SDR 21	SDR 33	SDR 51
Liner WT	42.9mm	13.0mm	20.6mm
Final ID with liner	794.0mm	373.2mm	956.3mm
100-year Gap bridging	fully-structural (infinite)	80mm interspace	20mm interspace
100-year Void spanning	fully-structural (infinite)	195mm void	206mm void
Collapse resistance	2.41 bar	7.9 bar	2.83 bar
Hydraulic performance	0.012bar* less head loss	0.07bar* more head loss	0.002bar* less head loss

Table 1. Design comparison different lining projects.

5.1 Hydraulic design

* Values in table 1 are hypothetical for comparison only and are not actual project flow rates, without detailed consideration of the total length of pipeline, nor any changes in elevation. The head losses have been calculated using the Hazen Williams equation, Equation 2 below.

$$h = 10.67.\left(\frac{q^{1.85}}{c^{1.85}.d_h{}^{4.8655}}\right) \qquad [2]$$

h head loss per unit pipe length (m_{H20}/m)
c Hazen Williams coefficient for the pipeline material
q flow rate (m^3/sec)
d_h pipe inside diameter (m)

6. CONCLUSIONS

- There are several techniques for installing PE 80 and PE 100 liners offering everything from fully-structural to semi-structural across the full SDR and size ranges
- Some lining technologies can install very long continuous lengths
- The three key design considerations for stand alone pressure pipes are only relevant for fully structural liners without structural support
- If a host pipe is in good condition retaining hoop strength, with minimal leakage at eg. joints and pinholes a semi-structural interactive liner can often be used
- For semi-structural interactive liners different design criteria are required
- Gap bridging and void or hole spanning capability can be designed for a 100-year lifetime
- Collapse resistance is enhanced by support from the host for tight-fitting liners over pipes or loose liners
- Liners are only suitable for applications where dissolved gas pressures are low, unless vented
- Hydraulic performance can be improved significantly even with a full pressure liner, though thinner liners provide greatest benefit
- Lower material costs, shorter welding times, reduced equipment spread, long installation lengths and navigation of modest bends make some thin wall semi-structural liners very economically attractive

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Design of Liners in Germany according to DWA-A 143-2

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1. INTRODUCTION

The design concept for liners in Germany presented here is based on Worksheet A 143-2 of the DWA - Deutsche Vereinigung für Wasserwirtschaft, Abwasser und Abfall e.V. (German Association for Water, Wastewater and Waste) (DWA) and was published in 2015. It is a further development of Code of Practice ATV-M 127-2 from 2000, which for the first time provided the user with dimensioning aids for liners that already took into account special features such as imperfections of the liner, long-term behaviour of the material or contact pressure problems between host pipe and liner. A revision and adaptation became necessary because of the introduction of the partial safety factor concept and some other new regulations and designations for traffic loads.
The calculations are supported by examples and tabulated coefficients, so that manual calculations are also possible for standard cases. The design rules used apply to construction and operating conditions as well as host pipe-soil systems with a stability > 1. Required data for the static calculation can be provided via a form (Annex G). Compared to its predecessor, the new Code of Practice DWA-A 143-2 now also contains the following additions:

- Updating the table for the material properties
- Partial safety factors
- Indexing for characteristic values and design values
- Definition of equivalent circles for egg profiles for stability and stress verification
- Notes on the application of calculation methods such as the FE method
- Extended information on the approach to imperfections
- Eccentricity of the host pipe joints

Within the framework of the written explanations given here and the associated lecture, the most important principles and key points from the author's point of view are dealt with, which are contained within the framework of the calculation of pipe liners with circular profile in Germany with the A 143-2. Of course, not all possible boundary conditions and variations can be considered. Consequently this presentation is only a small excerpt of what this very extensive set of rules has to offer.

2. Host pipe condition (HPC)

Damage to existing pipes usually becomes apparent when sewer inspections are carried out. These can take various forms (e.g. cracks or faults) and must first be assessed, as not all damage always requires the use of a liner. If the choice is made to use a liner in the case of leaks and cracked pipes, the selection of a suitable static system for the condition is of central importance. Ideally, information on the static calculation of the host pipe and the surrounding soil as well as the installation conditions is available, which can provide important information on boundary conditions. Unfortunately, this is not always the case. The static systems are represented in A 143-2 by the three host pipe condition (HPC) - I to III.

Host pipe condition (HPC) I:
The host pipe itself is still stable and not cracked. However, there are leaks in the pipe joints or the wall. The liner is stressed by external water pressure.

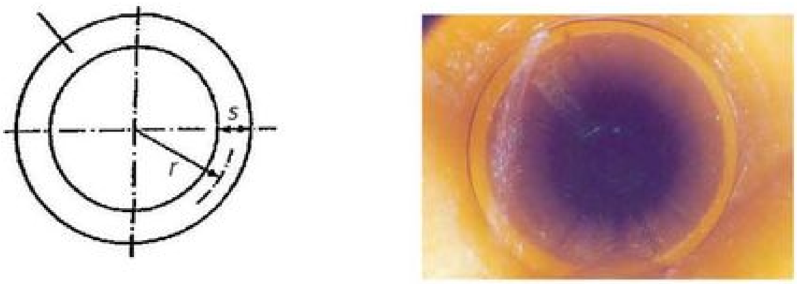

Figure 1: Host pipe condition I [DWA-A 143-2].

Host pipe condition (HPC) II:
The host pipe is cracked and leaking. However, the pipe deformation is low ($\delta_V < 6\%$). The pipe-bottom system is capable of bearing alone in the long term. The liner is stressed by external water pressure and must adapt to the ovalisation of the host pipe.

Figure 2: Host pipe condition II [DWA-A 143-2].

Host pipe condition (HPC) III:
The host pipe is clearly cracked and leaking. There is significant pipe deformation ($\delta_V > 6\%$). The pipe-soil system is no longer sustainable on its own in the long term. The liner is stressed by external water pressure, must adapt to the ovalisation of the host pipe and bear part of the earth and traffic loads of the host pipe.

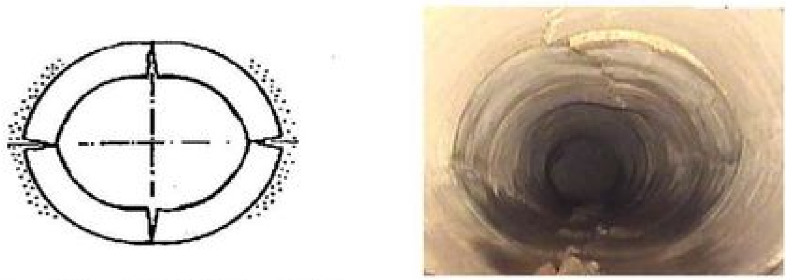

Figure 3: Host pipe condition III [DWA-A 143-2].

The definition of host pipe conditions is not always simple or clear. In particular, when differentiating between HPC II and HPC III, it is not only a matter of the ovalisation, which automatically changes into HPC III at >6%, but other boundary conditions must also be taken into account. Table 1 provides a summary of the differentiation.
For example, low cover heights (<1 m or < d_a +0,1 m) and/or changing superimposed loads are problematic, as this can lead to a concentration of loads at the apex of the host pipe. Widening of gypsum plaster pasted over a crack is a sign of an increase in deformation. Cavity formation outside the pipe due to infiltration can lead to a displaceable pipe-soil system, or the recalculation of the existing static system results in an utilisation factor of >1. If one of these cases already occurs, HPC III would already have to be applied (even if boundary conditions partly favour others).

Table 1: Criteria for the delimitation of host pipe conditions II and III [DWA-A 143-2, Table 11].

Criteria for old pipe conditions II and III			
	Indicator	II	III
1	Joint ring deformation GR,v (ovalisation)	≤ ca. 6%	> ca. 6%
2	Overlapping height	high	low
3	Traffic load effect	low	high
4	Load change (e.g. increase in surcharge)	no	yes
5	Plaster marks open	no	yes
6	Verification $q_{v,d}$ / crit $q_{v,d}$ (e.g. according to Annex F)	≤ 1	> 1
7	Cavity in the soil through infiltration	no	yes
8	Old pipe soil system with liner displaceable under external water pressure	no	yes

3. Safety concept with partial safety factors

With the introduction of the Eurocode and the partial safety factor concept, the old global safety factor concept of Code of Practice ATV-M 127-2 was replaced. The partial safety factor concept is based on the fact that the effects (loads) in the system are increased by a safety factor. At the same time, the resistances (material parameters) are reduced by a safety factor. The amount of the respective partial safety factor depends largely on the type of effect (permanent or variable) and the resistance (type of material). For example on effects, permanent loads receive a smaller safety factor than variable loads:

$\gamma_{F,G} = 1.35$ [-] (permanent loads) [1]
$\gamma_{F,Q} = 1.50$ [-] (variable loads) [2]

In terms of resistances, more homogeneous or factory-produced materials receive lower partial safety factors than those that were produced on site and tend to be of more heterogeneous composition (e.g. concrete):

$\gamma_M = 1.35$ [-] (liner, made on site) [3]
$\gamma_M = 1.50$ [-] (concrete, host pipe) [4]

In A 143-2, there are many more partial safety factors. The aim here is a more economical utilisation of the system with better consideration of the effects and resistances. However, the resulting safety level is generally comparable with the old global safety level. To indicate whether the respective values are characteristic values without a safety factor, an index "k" was introduced; design values with a safety factor are given an index "d".

4. Load and deformation of the host pipe

As the pipe is cracked in host pipe conditions II and III, deformations of the host pipe can occur depending on the vertical load q_v and the horizontal load q_h.
The choice of the load concentration factors λ_R and λ_B depends on whether the pipe has already cracked before rehabilitation and therefore behaves more or less like a flexible pipe, or whether it is assumed that the pipe only cracks after rehabilitation and is initially intact:

$q_v = \lambda_R \cdot \gamma_B \cdot h + P_T$ [kN/m²] [5]

$q_h = \lambda_B \cdot K \cdot \gamma_B \cdot h$ [kN/m²] [6]

with:

γ_B = Weight of soil [kN/m²]
h = Overburden height of the soil above the pipe [m].
P_T = Traffic load from surface at depth h [kN/m²].
K_2 = Horizontal earth pressure coefficient in the pipeline zone [-].
λ_R = Load concentration factor over the pipe [-], cracked/uncracked = 0.75 / 1.5
λ_B = Load concentration factor next to the pipe [-], cracked/uncracked = 1.08 / 0.83

If the surcharge load of the pipe $q_{v,d}$ and the modulus of elasticity E_2 in the pipe zone are known, the initial deformation δ_0 of the old pipe (see Figure 3) and the critical design load $q_{v,d}$ (see Figure 34) can be calculated via the measured joint ovality of the host pipe $\omega_{GR,v}$ with the help of diagrams from Annex F of A 143-2. Figure 4 and Figure 5 show examples of corresponding curves for soil group 2.

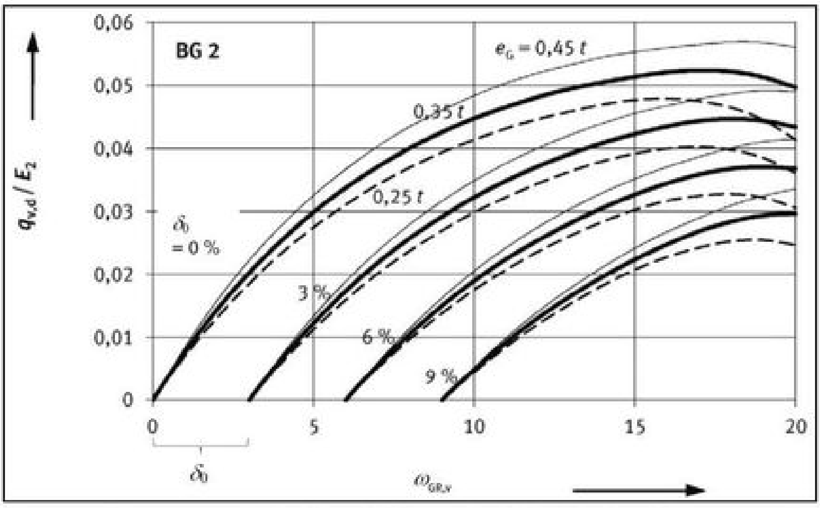

Figure 4. Load-displacement curves of the fourfold cracked host pipe for $q_{v,d}$, soil group G2, K_2 = 0.3 [DWA-A 143-2, Diagram F.3].

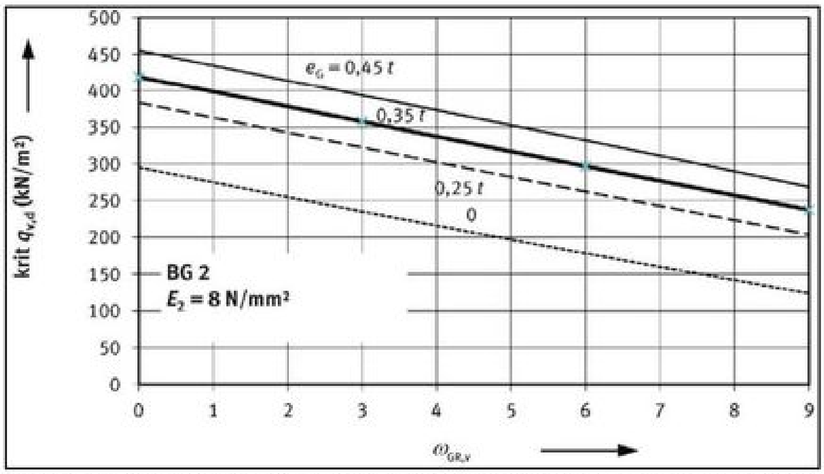

Figure 5. Critical vertical loads of the fourfold cracked host pipe, soil group G2, K_2 = 0.3; E_2 = 8 N/mm² [DWA-A 143-2, Diagram F.4].

The diagrams in Annex F contain values for the soil groups G1 to G3 with the associated horizontal earth pressure coefficients K_2 of the pipe zone and the angle of internal friction φ '. The system of soil groups and their parameters is derived from worksheet A 127 "Static calculation of sewers and pipes" (in trenches).
In the diagrams, the pivot point of the cracked four-joint system is also taken into account by a condition-dependent shift of the position of the pivot point within the joint.
This off-centre displacement of the pivot point in the joints is called joint eccentricity e_G (see figure 6) and is defined as follows:

$e_G = 0.45 \cdot t$ (good condition of the host tube transom with joints on the outside)
$e_G = 0.35 \cdot t$ (standard case)
$e_G = 0.25 \cdot t$ (severely damaged host pipe fighting with joints further inside)
$e_G = 0.00 \cdot t$ (centric joints, e.g. in the absence of grout in masonry).

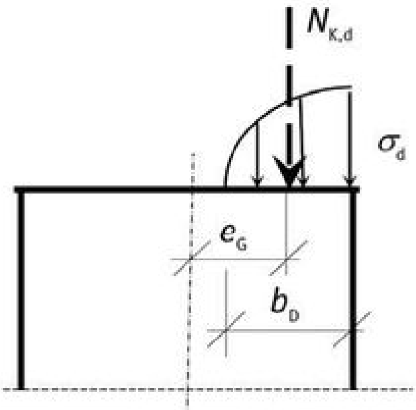

Figure 6. Parabolic stress distribution in the contact zone of longitudinal transom cracks [DWA-A 143-2].

5. Stress calculation in the liner

As already in the former Code of Practice A 127, the proportions for the design values of normal force and bending moments are determined from tabulated coefficients or diagrams, the design value of the external water load and the mean liner radius for the respective case.

$N_{pa,d} = n_{pa} \cdot p_{a,d} \cdot r_L$ [kN/m] [7]

$M_{pa,d} = m_{pa} \cdot p_{a,d} \cdot r_L^2$ [kNm/m] [8]

with:

n_{pa} = normal force coefficient, [-]
m_{pa} = moment coefficient [-]
$p_{a,d}$ = Design value of the external water pressure [kN/m²].
r_L = mean liner radius [m]

For the host pipe conditions I and II, the normal force components are determined with constant coefficients, which are on the safe side. For the host pipe condition III there are further diagrams (Annex E) which take into account the additional influence of earth and traffic loads $q_{v,d}$.
The stress calculation for the outer and inner side of the liner is then done by adding the determined stress components according to the following formula:

$\sigma_{i,d} = N_{pa,d} / A + \alpha_{k}, \cdot M_{pa,d} / W$ [kN/m²] [9]

$\sigma_{a,d} = N_{pa,d} / A - \alpha_{k}, \cdot M_{pa,d} / W$ [kN/m²] [10]

with:

A = Cross-sectional area of the liner [m²].
W = moment of resistance [m³]

For the stress verifications, the determined maximum occurring tensile and compressive stresses in the liner are ultimately compared with the design values of the reduced long-term characteristic values for the flexural tensile strength and compressive strength of the liner material as resistance:

$\max \sigma_{Z,d} / \sigma_{Z,d} \leq 1{,}0$ [-] [11]

$\max \sigma_{D,d} / \sigma_{D,d} \leq 1{,}0$ [-] [12]

with:

$\max \sigma_{Z,d}$ = maximum bending tensile stress on the liner [kN/m²].

$\sigma_{Z,d}$ = permissible bending stress of the liner [kN/m²].
max $\sigma_{D,d}$ = maximum compressive stress on the liner [kN/m²].
$\sigma_{D,d}$ = permissible compressive stress of the liner [kN/m²].

5. Load combinations

The load determination for the calculation of a liner in host pipe condition I and II is only oriented towards the calculation of the safety against stability failure (buckling) and can - with the exception of the load case water filling or internal pressure - in principle be carried out in one calculation step. However, when calculating a liner in host pipe condition III, several load cases and parameter combinations must be checked. A 143-2 presents these combinations in tables 12 and 13. They are summarised below in Table 2.

Table 2: Load combinations HPC I to III [DWA A 143-2, Tables 12 and 13].

Effect		Annular Gap	Dead Weight	Water Pressure	Earth- & Traffic Loads	Warming	Cooling	Water Filling	Internal Pressure
OPC	LC	[%]	[g]	[P_a]	[q_v]	[ϑ+]	[ϑ-]	[W]	[P_i]
I, II	1a	0,5	X	X	---	---	(X)	---	---
I, II	1b	0,5	X	X	---	(X¹)	---	---	---
I, II	2	0,5	X	---	---	---	---	X²	X³
III	2	0	X	---	X	---	---	---	---
III, Interaction	3a	≥ 0,5	X	X	X	(X¹)	---	---	---
III, Interaction	3b	0	X	X	X	(X¹)	---	---	---
III	4	≥ 0,5	X	---	---	---	---	X²	X³

Notes:
1) Use reduced E-Modulus for heating
2) This can become decisive for egg-shaped profiles in connection with larger gap widths.
3) Check egg profiles even at p_i < 0.5 bar if necessary.

It is not possible to say in advance which load combination is decisive, as this is largely dependent on the existing groundwater level and the depth of the pipe. The reasonableness of the load combinations can best be explained using the following procedure:

1. First, a calculation is made in HPC III under LK 3a with all loads from external water pressure and earth and traffic loads). The calculation takes into account the annular gap in order to generate the largest possible bulge load from water pressure on the one hand and to take into account the influence of the earth and traffic loads (under buoyancy) on the other.
2. In LC 3b, the annular gap is not taken into account in order to give priority to the influence of earth and traffic loads in direct contact with the liner in the combined action.
3. In LC 2, groundwater and buoyancy are excluded and only the earth and traffic loads are taken into account without the annular gap.
4. In LC 1, a comparative calculation is then carried out in HPC II with annular gap and water load for comparison.

The load cases of heating and cooling of a liner during installation are not considered in this representation. For heating, however, reduced E-moduli are to be expected. The load case of internal pressure is usually unproblematic for circular profiles, but can be important for egg-shaped profiles.

6. Stability failure (buckling)

Liners are slender structures that are sometimes exposed to high stresses from external water pressure. Therefore, they must also be checked for stability failure due to buckling in the course of the calculation. In A 143-2, this liner failure is described by the bedded pipe-in-pipe model according to Glock, 1977, which is shown in Figure 7.

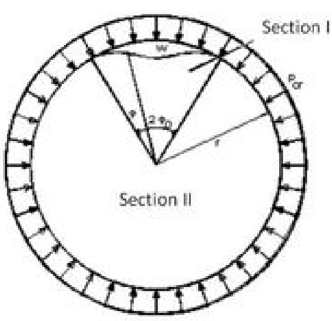

Figure 7. Pipe-in-pipe model [Glock, 1977].

The critical pressure that can be absorbed by the system is described by the following equation:

$p_{krit} = k_{v,s} \cdot \alpha_{krit} \cdot S_{L,d}$ [kN/m²] [13]

with:

$k_{v,s}$ = Reduction factor to take imperfections into account [-].
α_{krit} = Breakdown coefficient to take into account the bearing of the liner [-].
$S_{L,d}$ = Pipe stiffness with E-modulus for long-term behaviour [kN/m²].

The breakdown coefficient for a free pipe is $\alpha_{krit} = 3.0$. For a bedded pipe it depends on the ratio r/t and results in values of approx. $35 < \alpha_{krit} < 60$ for ratios of $25 < r/t < 50$. The buckling stiffness of a fully bedded pipe is thus approx. 10 to 20 times higher than that of a free pipe in these areas of application.

Since a perfect circular ring at a constant water pressure on the outside represents a very resistant structure, which, however, does not occur in this way in the sewer, the following types of imperfections in the liner must still be taken into account when calculating the safety against stability failure, which are shown graphically in Figure 8. The values given are standard values:

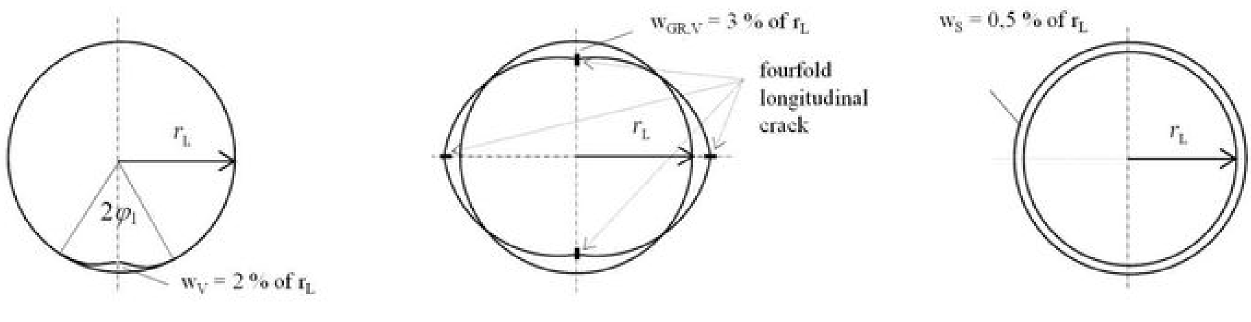

Figure 8. Local imperfection w_v, joint ring deformation $w_{GR,v}$ and annular gap w_s [DWA-A 143-2].

The **local imperfection** w_v takes into account deposits in the pipe bottom and deviations from the nominal shape or structural variations of the E-modulus and the liner thickness. It is also necessary to trigger the stability case.

A **joint ring deformation** $w_{GR,v}$ occurs with cracked host pipe due to the deformation.

An **annular gap** w_s occurs in liners due to shrinkage of the resin content in the liner or the backfill material, changing internal profile dimensions, assembly inaccuracies, etc.

Each individual imperfection is assigned a reduction factor < 1.0, which leads to a reduction of the critical buckling load on the resistance side These individual factors are multiplied with each other and combined to a total reduction factor $k_{v,s}$. For the standard case of a pipe liner, the A 143-2 provides the diagram shown in Figure 9 for determining the total reduction factor $k_{v,s}$ as a function of the joint ring deformation $w_{GR,v}$. For other liner shapes (e.g. egg profiles) or materials or manufacturing processes, there are separate reduction diagrams for each imperfection.

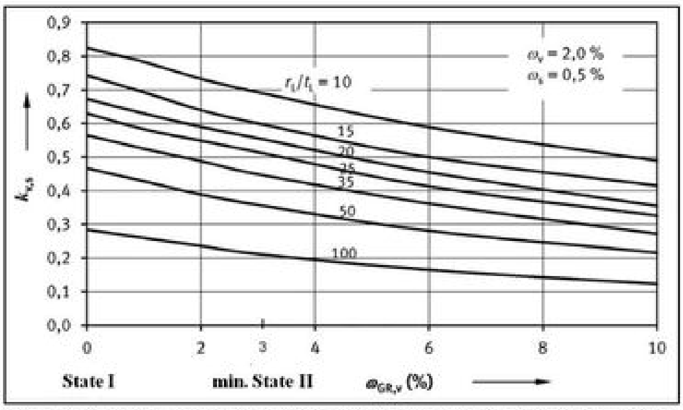

Figure 9. Diagram for determining kv,s as a function of the joint ring deformation $w_{GR,v}$ [DWA-A 143-2].

Inserting the formulas for the breakdown coefficient and the pipe stiffness into equation 13 gives the following expression in A 143-2:

$$p_{krit} = k_{V,S} \cdot 2{,}62 \cdot \left(\frac{r}{t}\right)^{0{,}8} \cdot \frac{E}{12 \cdot (1-\nu^2)}\left(\frac{t}{r}\right)^3 \quad [\mathrm{kN/m^2}] \qquad [14]$$

By cleverly shortening the terms, equation 14 can also be written as follows:

$$p_{krit} = k_{V,S} \cdot 1{,}0 \cdot \frac{E}{1-\nu^2}\left(\frac{t}{D}\right)^{2{,}2} \quad [\mathrm{kN/m^2}] \qquad [15]$$

For the proof of the buckling resistance of the liner, the design value of the acting water pressure $_{pa,d}$ is compared with the critical buckling pressure pkrit resulting from the stability calculation, analogously to the stress proofs:

$$p_{a,d} / \mathrm{krit}\ p_{a,d} \leq 1{,}0\ [-] \qquad [16]$$

with:

$p_{a,d}$ = Design value of the acting water pressure on the liner [kN/m²].
krit $_{pa,d}$ = Critical bulge pressure of the liner [kN/m²].

7. Summary

In the context of this paper, the basic functionalities of the worksheet DWA-A 143-2 used in Germany were explained using the example of a circular pipe liner. The use of the worksheet also enables manual calculations for standard cases with the help of tables and diagrams. The dimensioning of liners is fundamentally oriented towards the host pipe conditions I to III, which provide the basis for the selection of the static system. Since the load scenarios can be different and the worst case is not always clear, liners (especially in HPC III) are calculated in various load combinations. The maximum compressive and tensile stresses in the liner as well as the safety against stability failure (buckling) are calculated. The cracked host pipe is also calculated for sufficient pressure transfer in the joint rings. The calculation of different liner types and shapes is possible by adjusting the boundary conditions.

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Session 2 – Pressure pipe rehabilitation

Paper N°139

Response of a cured in place liner in cast iron water pipe to joint expansion due to permanent ground deformation or seismic wave

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Abstract The response of segmented pipelines to permanent ground deformation as well as seismic wave are briefly discussed. Permanent deformations can be due to earthquake as well as tunnel boring or voids collapse. Ground strain typically varies from 0.1% to 1% and the corresponding joint opening varies from few mm to few cm. Unlike ductile iron pipes, cast iron pipes with lead caulked joints have low joint expansion capacity since leakage may occur at joint opening as low as 2 mm. The response of a cured-in-place pipe (CIPP) liner as it spans a joint expanded is then examined through analytical approach and finite element axisymmetric model. The effects of friction between the liner and the old pipe as well as the internal pressure and the material properties of the liner (Poisson's coefficient and Young modulus) are investigated. It is shown that the axial stress increases with the pressure, the coefficient of friction and the modulus but that it decreases with the Poisson's ratio. The Poisson's ratio has here a positive effect because it decreases the contact pressure as the liner stretch. It is also shown that the axial stress can easily exceed the hoop stress of the free tube under internal pressure even for a displacement as small as 1% of the host pipe diameter. It is concluded that only a loose-fitting installation can guarantee the resistance of the liner.

Introduction

The forces induced on buried pipes due to ground movements are very different from those due to surface loads or gravity. Buried pipes follow the movements of the ground with little inertia effect (the movement of the pipe is in phase with that of the ground without any resonance effect, this fact allows the soil-pipe interaction problem to be reduced to a static one). Mechanically, it is about a problem of imposed displacement and not of imposed force. The ground movements can be permanent, for example the settlement due to the digging of a tunnel or a trench (Argyrou, 2016) or variable due to seismic waves.

Continuous pipelines may break in tension or buckle in compression, segmented pipes tend to break at the joints (axial pull-out in tension, crushing of bell and spigot in compression). Small diameter segmented pipelines (ratio length/diameter > 10) may also break, away from the joints, in bending due to ground curvature.

Continuous steel pipes and ductile iron pipes (with locked joints) may resist to a tensile strain of 3 to 5%, PEHD pipes is capable to accommodate large strains higher than 10%, conversely, gray cast iron pipes will break at a tensile strain of 0,2%. It is commonly accepted that pipes made of flexible materials (PE, PVC...) have superior seismic performance than rigid/brittle materials like concrete pipes.

For segmented cast iron pipes with lead caulked joints it is commonly assumed that the joint opening that causes a leak is approximately 50% of the length of the joint depth. However, on the basis of our own observations on old gray cast iron pipes (around 120 years old), we believe that some leakage may occur for relative displacements as small as 1 to 2 mm.

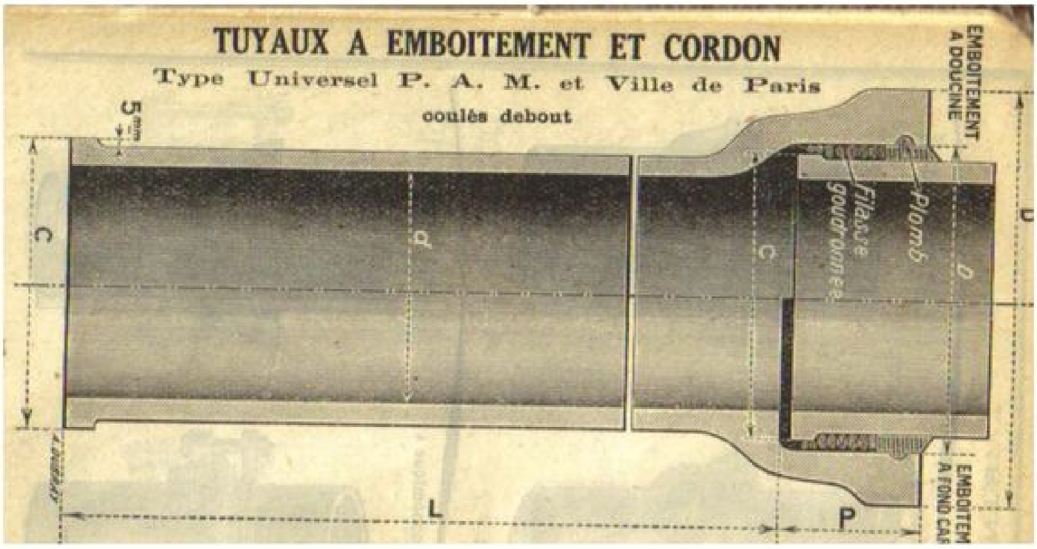

Figure 1. Gray cast iron pipe with bell and spigot and with lead caulked joint. Extract from the 1920 Pont-à-Mousson catalog.

The use of plastic pipes (close-fit liners or slip-liners) appears to be a relevant solution to secure segmented cast iron pipes with respect to the risk of leakage under the effect of seismic waves or differential settlements.

The standard ISO 11295 proposes a structural classification of pressure pipe liners in four classes from class A (fully structural) to class C (non structural).

In the Table 16 we read: "Class A pipe liners can survive internally or externally induced (burst, bending or shear) failure of host pipe". In the text below, class A liner is defined as an "Independent pressure pipe liner" which "is, by definition, capable on its own of resisting, without failure, all applicable internal loads throughout its design life, without relying on the existing pipeline for radial support. When tested independently from the host pipe, it should exhibit long-term 50-year internal pressure strength equal or greater than the PFA". However, this second definition is not equivalent to the first one, in fact, it has not been demonstrated that the capacity to withstand internal pressure is sufficient to survive the rupture of the host pipe. In addition, it is important to emphasize that pressure thrusts at bends and ends are still transmitted to the host pipe, so it is not correct to pretend that the pipe liner is "independent" to the host pipe, in many installations the liner continues to transmit more than 95% of the internal pressure to the host pipe internal.

A Class A pressure pipe liner can be a loose-fitting or close-fitting installation, in this paper, it is assumed a close-fitting installation.

Ground strain due to wave propagation

Newmark (1967) developed a simplified procedure to estimate the ground strain. He assumed that the soil motion is represented by a single sinusoidal wave:

$$u(x,t) = \mathrm{d} \cdot \sin \omega \left(t - \frac{x}{c}\right)$$

Where d is the displacement amplitude, and c is the velocity of the seismic wave.

The ground strain in the direction of wave propagation is given by:

$$\epsilon = \frac{\partial u}{\partial x} = -\frac{\omega d}{c} \cdot \cos\omega\left(t - \frac{x}{c}\right)$$

Whose maximum value is:

$$\varepsilon_{max} = \frac{V_m}{c}$$

where:

$V_m = \omega d$ is the peak horizontal ground velocity.

The maximum ground curvature that is the second derivative of the transverse displacement with respect to x is given by:

$$\chi = \frac{\partial^2 u}{\partial x^2} = -\frac{\omega^2 d}{c^2} \sin\omega\left(t - \frac{x}{c}\right)$$

Whose maximum value is:

$$\chi_{max} = \frac{a_m}{c^2}$$

where:

$a_m = \omega^2 d$ is the peak soil acceleration.

The soil motion parameters Vm and a_m can be obtained from earthquake records.

The velocity of seismic shear waves depends on the nature of the soil and the thickness of the layers. Table 3.1 of the EN 1998-4 (Eurocode 8, 2005) gives ranges of values, for example:

- Deep deposits of dense or medium dense sand, gravel or stiff clay 180 – 360 m/s;
- Deposit of loose to medium cohesion less soil: < 180 m/s.

The shear wave velocity of 150 m/s is characteristic of moderately dense surface soil layer.

The peak horizontal ground velocity depends on the earthquake magnitude, the hypocentral distance and the site amplification factor and may typically varies from few cm/s to 50-80 cm/s with a representative value of 30 cm/s (O'Rourke 1999). The corresponding ground strain for a shear wave velocity of 150 m/s is $\varepsilon_{max} = 0.3/150 = 0.2\%$.

Response of segmented pipelines to ground strain

Ground strain can create expansion/contraction of pipe segments and joints. Since the axial stiffness of the pipe segment is much higher than that of the joints, we can assume that the displacements are concentrated at the joints and that the joint displacement is approximately equal to the product of the ground strain times the pipe segment length.

$$g = \varepsilon_{max} \cdot L_{pipe}$$

The length of buried gray cast iron pipes varies from 4 to 6 m. assume that the pipe segment length is 6.0 m.

For a ground strain of 0.2% the expected joint opening is 12 mm.

Response of a close-fit liner installed in a segmented pipeline to joint expansion

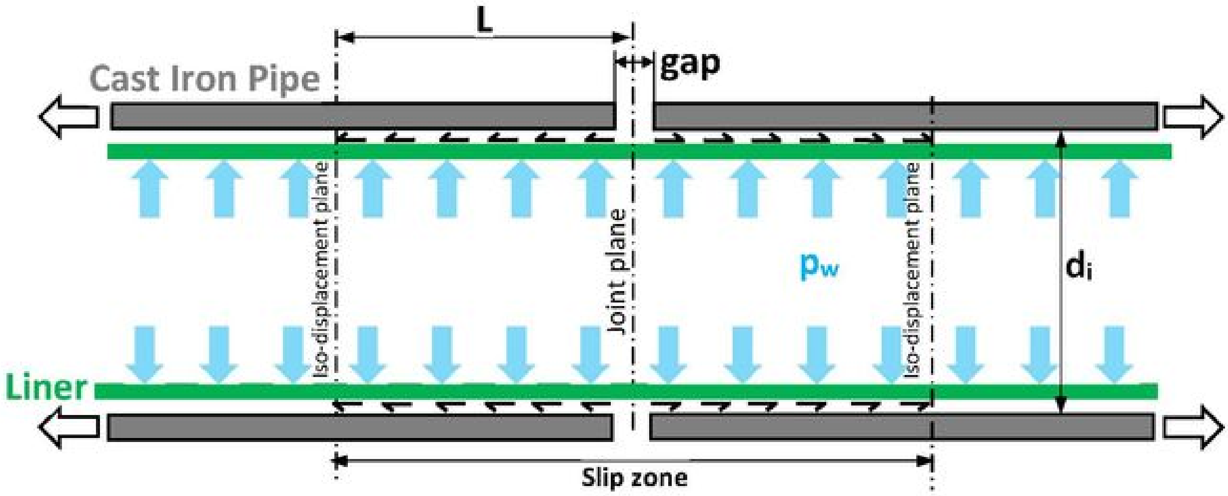

Figure 2.

The expansion of the joint causes the 2 pipe segments to slip and the development of shears at the liner-pipe interface (Figure 2.).

Shears create axial force in the liner and then axial strain and axial displacement. Axial displacement increases with the distance from the joint to the iso-displacement plane where the displacement of the lining and the pipe are equal (the relative displacement between the liner and the host pipe is zero) and where the fiction is cancelled.

It is assumed that shear stress at the interface liner/inner pipe is proportional to the contact pressure:

$$\tau(x) = \mu \cdot p_c(x)$$

where μ is the friction coefficient.

Mike Brown (Brown 2014) got a simple solution for the maximal axial stress as a function of the gap (joint opening) however this solution is only exact for a zero Poisson's ratio. We therefore resumed the calculation but with a non-zero Poisson's ratio. The new solution has been calculated with Lame's equations. We give the main results without detailing the derivations.

Length of the sliding section (see also Fig. 2)

$$L = \left(\frac{\eta}{2} + \frac{1-e^{-\eta-0.2\eta^2}}{\eta}\right) L_0$$

Where:

$$L_0 = \sqrt{\frac{gtE}{\mu p_i (1-\nu^2)}}$$

$$\eta = L_0 \frac{\mu\nu}{r}$$

$L \geq L_0$

For $\nu = 0$, $L = L_0$

Axial stress:

For $x \leq L$:

1) ν > 0

$$\sigma_{33}(x) = \frac{p_i r}{\nu t}(1 - e^{-\frac{\mu\nu(L-x)}{\mathrm{r}}})$$

The maximum axial stress is given by:

$$\sigma_{Max} = \sigma_{33}(0) = \sigma_L(1 - e^{-\frac{\mu\nu L}{\mathrm{r}}})$$

Where:

$$\sigma_L = \frac{p_i r}{t\nu}$$

When g → ∞ , $\sigma_{max} \to \sigma_L$

2) ν = 0

$$\sigma_{33}(x) = \frac{p_i \mu}{t}(L - \mathrm{x})$$

The maximum axial stress is given by:

$$\sigma_{Max} = \sqrt{\frac{gE\mu p_i}{t}}$$ (Brown' solution)

Contact pressure:

For x ≤ L:

$$p_c = p_i\left[e^{-\frac{\mu\nu(L-x)}{\mathrm{r}}} - (1-\nu)\frac{t}{r}\right]$$

When g → ∞ , $p_c(0) \to 0$

For x ≥ L

$$p_c = p_i\left[1 - (1-\nu)\frac{t}{r}\right]$$

For ν = 0

$$p_c = p_i\left[1 - \frac{t}{r}\right]$$

Axial displacement:

For x ≤ L:

1) ν > 0

$$u(x) = g\frac{r^2}{\mu^2\nu^2 {L_0}^2}\left[\frac{x\mu\nu}{\mathrm{r}} + e^{-\frac{L\mu\nu}{\mathrm{r}}} - e^{-\frac{(L-x)\mu\nu}{\mathrm{r}}} + \frac{tx\mu\nu^3}{\mathrm{ri}^2(1-\nu)}\right]$$

2) ν = 0

$$u(x) = \frac{\mathrm{pi}}{2\mathrm{E}t}(2L_0 x - x^2)$$

For x ≥ L:

$$u(x) = g/2$$

Comparisons with FEA

For the purpose of this comparison with FEA, we have considered a pipe with an internal diameter of 300 mm and a liner with a thickness of 4 mm.

The Young's modulus (in the short term since it is a fast phenomenon) of the liner is 10,000 MPa and its Poisson's ratio of 0.3. We also tested a Poisson's ratio of 0 (solution of Mike Brown).

The coefficient of friction liner / host pipe is equal to 0.4 which is a usual value for an old cast iron pipe.

The internal pressure is 0.6 MPa (6 bars). The hoop stress in the unconfined liner σ_H would be equal to 22.2 MPa.

The gap (opening of the joint) varies from 0 to 30 mm.

The calculations were carried out in symmetry of revolution (axisymmetric) using Ansys Mechanical version 17.2.

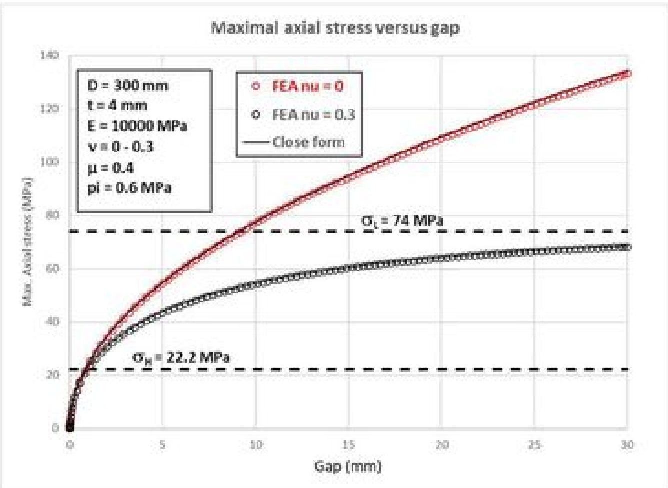

Figure 3. Maximal axial stress versus gap and Poisson's coefficient.

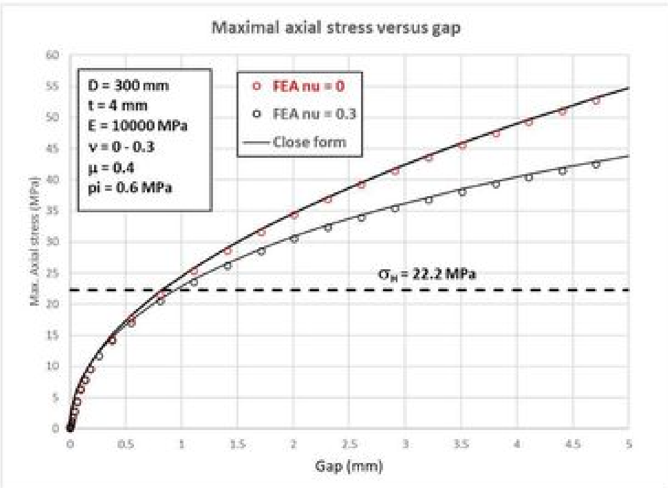

Figure 4. Detail of figure 3.

The Fig. 3 presents the evolution of the axial stress as a function of the gap and the Poisson's ratio. It can be seen that the axial stress is significantly higher for a zero Poisson's ratio than for a Poisson ratio of 0.3, and that the difference increases quickly with the gap.

The reduction in axial stress is due to the Poisson effect which causes the transversal contraction of the liner under the axial expansion. This transverse contraction decreases the contact pressure and so the shear stress.

It is also noted that when the Poisson's ratio is not zero, the axial stress converges towards the hoop stress of the unconfined liner divided by the Poisson's ratio (σ_L = 74 MPa).

It can also be seen from Fig. 4 that the axial stress exceeds the hoop stress of the unconfined liner when g exceeds 1 mm.

For our reference joint opening of 12 mm, the axial stress reaches 57 MPa a value 2.6 times higher than the hoop stress of the unconfined liner (22.2 MPa).

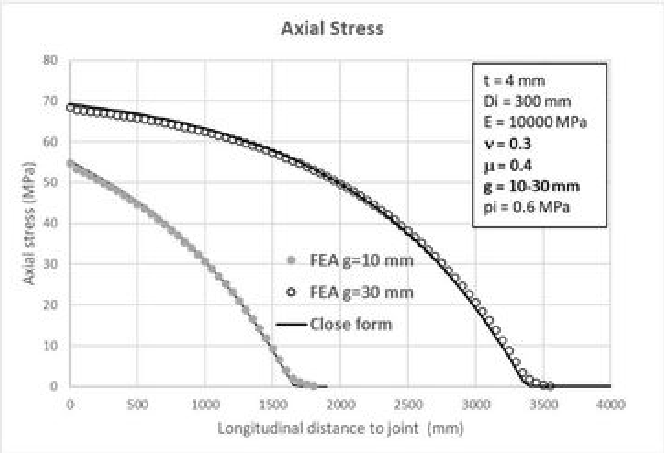

Figure 5. Axial stress versus longitudinal distance to joint and gap.

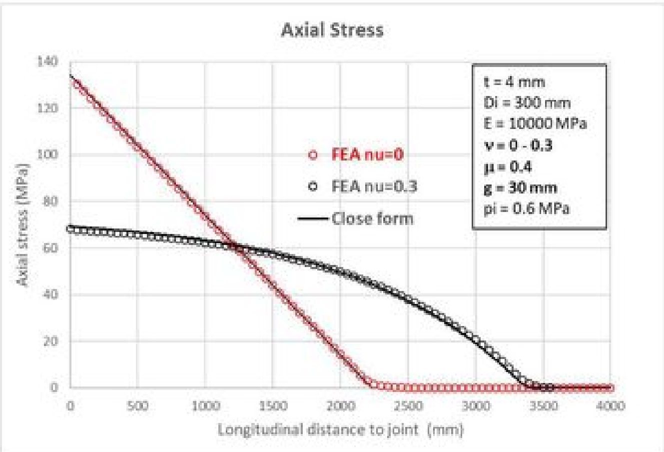

Figure 6. Axial stress versus longitudinal distance to joint and Poisson's coefficient.

Figures 5 and 6 show the evolution of the axial stress as a function of the distance from the joint. It is observed that the stress decreases from its maximum value at the level of the joint to zero at the end of the sliding zone. When the Poisson's ratio is equal to zero the maximum value is significantly higher, the decrease is linear and occurs over a shorter length.

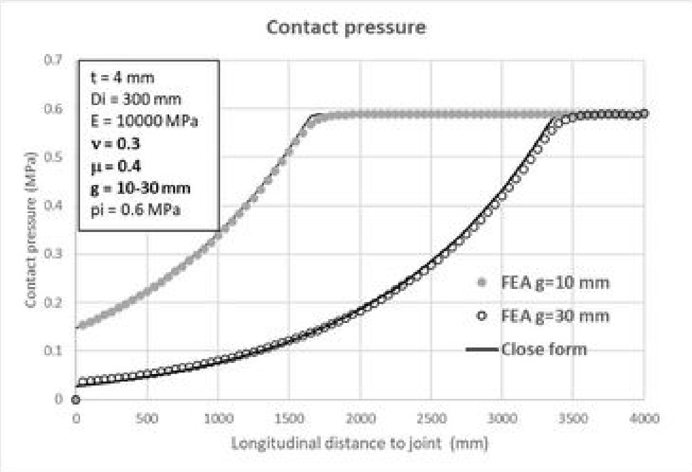

Figure 7. Contact pressure versus longitudinal distance to joint and gap.

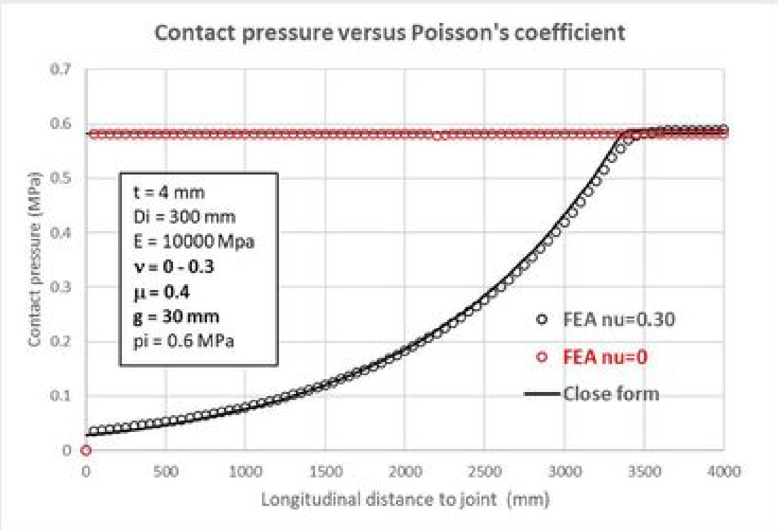

Figure 8. Contact pressure versus longitudinal distance to joint and Poisson's coefficient.

Figures 7 and 8 show the evolution of the contact pressure with the distance to the joint. It can be seen that it is minimum at the level of the joint and that it increases over the length of the sliding zone to almost reach the value of the internal pressure. Conversely, when the Poisson's ratio is zero, the contact pressure is constant.

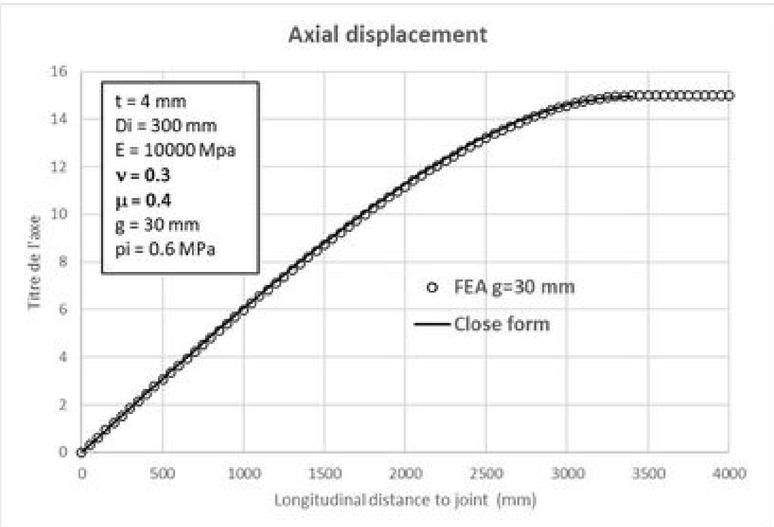

Figure 9. Axial displacement.

Conclusions

This paper presents a close form solution which allows to calculate the axial stress which appears in a close-fit liner installed in a segmented pipe during the expansion of a joint due to seismic wave. The analytical solution was established for an isotopic, elastic linear material and compares very well with finite element results. It is interesting to note that the Poisson's ratio has here a positive effect by reducing the contact pressure between the liner and the host pipe.

It is shown that the axial stress which appears in the liner during the expansion of a joint can be much higher than the hoop stress in the unconfined liner due to the internal pressure. Therefore, the resistance to internal pressure is not a sufficient condition to “survive internally or externally induced failure of host pipe” when the liner is in close contact with the host pipe. Only a loose-fitting installation can guarantee the resistance of the liner.

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A UNIQUE EXAMPLE OF CLOSE FIT LINING TECHNOLOGY FORTHE RENEWAL OF WATER PIPES ALONG THE BRIDGE "PONTE PUNTA PENNA" IN TARANTO

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ABSTRACT: The Project concerning the rehabilitation of 4 pressure pipes located inside the bridge "Ponte Punta Penna-Pizzone" in Taranto represents one of the most significant international examples in the field of close-fit lining technology for the rehabilitated lengths, the particularities of the building area and the construction difficulties. The bridge rests on 14 spans rising to 47m and holds four steel pipes of 1.200m, DN 500mm hanging inside the deck of the structure. Through these pipelines passes a water flow of 500 l/s ensuring the water needs of about 200.000 inhabitants. The rehabilitation through close-fit lining technology consists to insert into the old pipe a new tube in high-strength polyethylene, which is temporarily deformed and subsequently restored into the required shape and size to adhere perfectly (close-fit) to the inner wall of the existing pipe. The advantage consists in avoiding a significant reduction of the hydraulic section and in obtaining a new pipeline that guarantees its own structural resistance. The following 4 main issues had to be managed: the pipes are hanging inside the bridge without a continuous confinement; there was no possibility to modify the bridge creating new access points; the only access is placed about 9m above ground, as well as the winch location and a length of 1.200m was required. The major concern was the investigation of the force distribution throughout the entire length of the pipe, which would have determined the feasibility of the whole project. A shoring system, saddle steel plates and a steel carrier were used in order to avoid any damages or concentrated stresses on the existing pipe. All of these elements and the force distribution were calculated and verified from a structural point of view with a 3D FEM model. This was a very important step, because any damages or deformations would have blocked the new PE (polyethylene) pipe. 3 blocks composed by 20 PE pieces were prepared along the adjacent street, each of them 20m long, so that 400m long sections were welded and insert at a time. The pipe was brought up to the entrance point and helped by some support rolls. After the insertion, the connections were restored. It was immediately clear that a qualified team had to be identified with a strong innovative capacity and capable of ensuring a coordinated management of the project throughout all its development. The cost of the rehabilitation was 2.5 million euro and it was successfully completed in about 9 months.

1. CONTEXT OF THE PROJECT

The project, promoted by Acquedotto Pugliese S.p.A., concerning the renovation of four pressure pipes located inside the bridge "Ponte Punta Penna-Pizzone" in Taranto (Italy) represents one of the most significant international examples in the field of close-fit lining technology.
The Italian region, Puglia, is the only shareholder of the society Acquedotto Pugliese S.p.A that leads the Integrated Water Service in the entire region. It manages the water supply in different municipalities located in region Campania, as well as providing sub-contracted water up to 40% to the region Basilicata, in southern Italy. Acquedotto Pugliese cares about one of the most complex system in the country for drinkable water regarding its transport and distribution, characterized by different hydraulic devices, transport capacity and state of age - maintenance. The company is responsible for 25.000 km of pressure pipes (5.000 km for supply and 20.000 km for distribution), serves 250 districts and provides drinking water to 4 million citizens.

Fig.1 water network managed by the company Aquedotto Pugliese S.p.A, AQP.

The bridge rests on 14 spans rising up to 47m above the sea and holds four steel pipes of 1.200m in length, DN 500mm, hanging inside the deck of the structure. Through these 4 pipelines, a water flow of 500 l/s passes, providing the water needs of approximately 200.000 inhabitants; 40% of the population of Taranto. The pipelines were built at the same time as the bridge, which was inaugurated on 30 July 1977and it is one of the longest in Europe that connects two coasts separated by a sea, Figure 2.

Fig.2: Ponte Punta Pizzone

2. STATUS OF FACT AND PROJECT SOLUTION

The accurate inspection of the pipelines, carried out from AQP S.p.A. in the year 2010-11, showed a widespread and advanced state of oxidation of the pipes, especially in correspondence of the joints between pipe elements and between pipe elements and hydraulic devices. The inspection also pointed out a severe steel deterioration located at the drainage points of the rainwater entering from the overlying road structure. Figure 3 shows a section of the inside of the bridge.

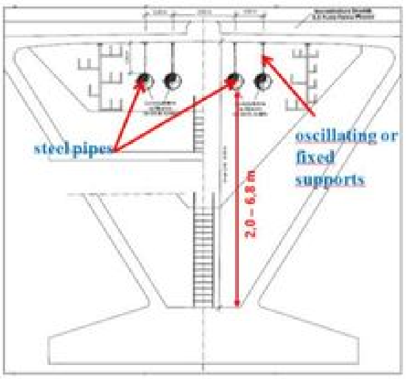

Fig.3: bridge section

The thickness measurements carried out, all along the pipelines, confirmed the advanced state of deterioration, where the reduction of the wall amounted to 3 up to 4 mm and locally even up to 6 mm, Fig.4.

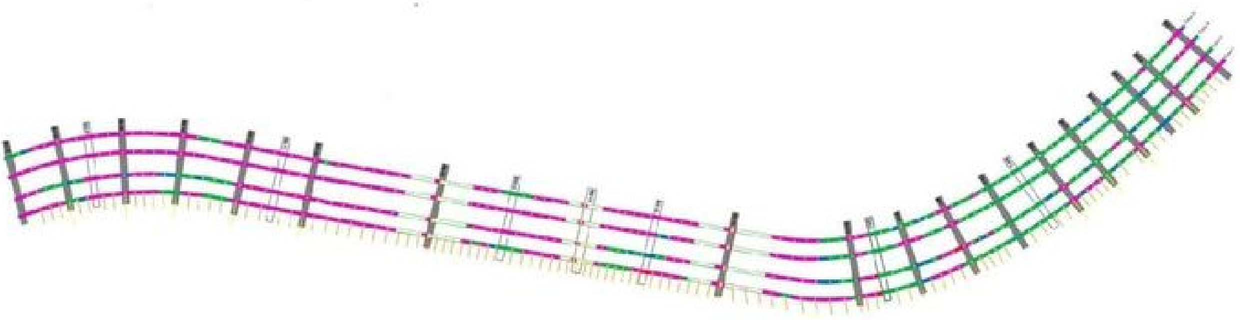

Fig.4: planimetric diagram of the pipes inside the bridge, with indication of the measured thicknesses

Pipe section with remaining thickness 7,5 - 8,0 mm

Pipe section with remaining thickness 6,0 - 7,5 mm

Pipe section with remaining thickness 4,2 - 5,9 mm

Pipe section with remaining thickness 1,8 - 4,0 mm

The pipe section is not measurable, there are no particularly conspicuous problems

It was considered necessary to design an intervention aimed at restoring the structural characteristics possessed by the pipes at the time of their construction, in order to guarantee the continuity and the quality of the water service. The rehabilitation had to fulfil the following essential needs and it was also influenced by logistical and environmental aspects:

- To guarantee the structural resistance and to give the pipes an adequate response to corrosive agents and environmental deterioration;
- To limit the pipeline off-duty time and meet minimum service levels;
- It was not possible to replace the existing pipelines from the surface, so as not to affect the structure of the bridge;
- The working environment inside the structure does not allow the transit of large mechanical vehicles;
- The presence of only two pedestrian access points to the deck of the bridge located at the opposite ends of the pipes.

After a careful evaluation, a NoDig Technology was chosen: Close-Fit Lining.
The rehabilitation is based on the insertion of a new tube, composed of high-strength polyethylene, into each of the four pipelines. The new pipe is temporarily deformed and subsequently restored into the required shape and size to adhere perfectly to the interior of the existing pipe. The advantage consists in the avoidance of a significant reduction of the hydraulic section and the obtainment of a new pipeline that guarantees its own structural resistance, independent of the support of the existing pipeline.
The advantages of using this trenchless technology compared to replacing the existing pipelines can be summarized as follows:

- Better resistance to corrosion with the same structural guarantees provided by the complex steel pipe + PE piping, resulting in an extension of the working life cycle by 80 years. The preservation of the existing steel pipeline can be subsequently guaranteed with ordinary maintenance interventions to be carried out from the outside, without the need to interrupt the water flow;
- Out-of-use times of the rehabilitated pipelines reduced to a minimum, resulting in less discomfort for the served population;
- Reduction of the risks for the whole work carried out by the workers inside the bridge;
- Reduction of the production of the waste and relative costs of about 470 tons of existing pipe to be disposed of;
- Reduction of environmental pollution due to the transport needed on site.

3. THE FINAL PROJECT

The minimum thickness of the new PE pipe to be inserted was dimensioned not only in relation to the maximum expected working pressure, but also by verifying the structural stability of the PE pipe itself.
The system was also verified in the case of serious corrosion phenomena on the existing pipe, leading to the detachment of the entire portion of the steel pipe between two adjacent supports, about 10,5m apart, as shown in the load scheme below

$$q = W_{water} + W_{PE} + W_{steelpipe}$$

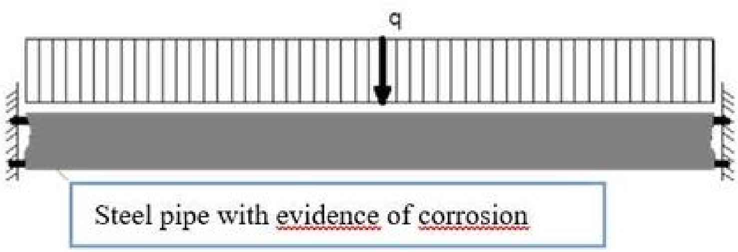

Fig.5: Steel pipe with evidence of corrosion

Subsequently, having considered a single point for the insertion and a single point of exit, the new PE pipeline was verified in relation to the pulling actions necessary for the insertion, as identified in the diagram, Fig.6. A crucial point was the study of the forces to which the pipelines and the bridge would have been exposed during the rehabilitation. The maximum friction force was expected to be registered at the point where the new pipe would have been inserted into the existing tube completely.

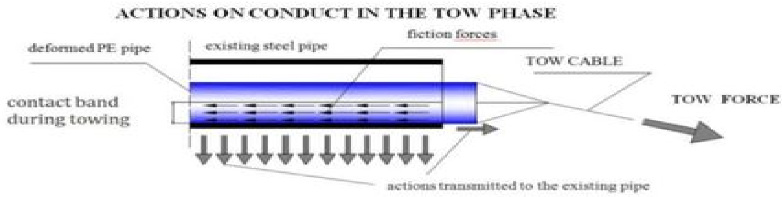

Fig.6: Forces distribution

The final project was drawn up by the technicians of Acquedotto Pugliese S.p.A. (Ing. G. Casamassima, C.D. Tria and A. Schinaia). The tender for the executive project and the realization of the intervention, for a sum of approximately € 2.500.000 was awarded to the company ROTECH Srl. The executive and constructive project was carried out by Studio Ing. Cimini of Taranto in collaboration with the same contractor and Ing. Dieter Schölzhorn of Studio Ing. Valdemarin (Bressanone).

4. IMPLEMENTATION STEPS AND TECHNICAL SOLUTIONS

The contractor's executive project involved the use of a particular type of close fit lining called Dyn-Tec, which during the execution phase involves the reduction of the diameter of the PE pipe by passing through a "Reduction Cone" (RIG). It also included the insertion into the existing pipe by means of the pull exerted by a winch installed at the opposite end of the insertion point, 1.200m away.

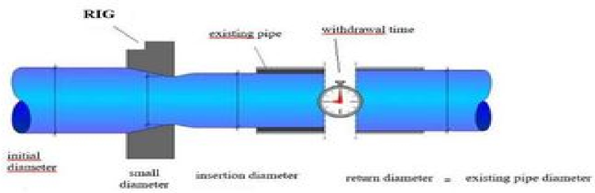

Fig.7: Reduction of the diameter

When the company approached this project, understood that it would face the following main issues:

- The pipes were hanging inside the bridge without a continuous confinement as it would be the case in an underground system;
- Due to the importance of the road structure, there was no possibility to modify the bridge creating new access points;
- The only access point was placed circa 9m above ground, as well as the winch location;
- A length of 1.200m was required, interrupted by two omega sections installed to deal with the thermo-dilatation of the pipe

The most important constraint was that there wouldn't be any forces acting on the bridge, as demanded from the ANAS, the institution that manages roads owned by the state, as well as providing for their ordinary and extraordinary maintenance. Therefore, the main concern was the investigation of the force distribution throughout the entire length of the pipe, which would have determined the feasibility of the whole project.
In addition to the expected working force, also the structural stability of the PE pipe and the water load had to be included in the considerations concerning the dimensioning of the minimum pipe thickness. So, the aim was to study the factors that could have created any forces that had to dealt with.
The maximum force required for the insertion of the 1.200m of PE pipe was equal to the action to obtain the reduction in diameter of the PE pipe, plus the action necessary to overcome the friction between the PE pipe and the steel existing pipe during the pulling phase for a total estimate of 80 tons. In order to contain the total pull force within 50 tons and therefore to limit the actions transmitted to the deck of the bridge, two devices were coupled to the RIG, so called "pushers" that allow to reduce the pulling force necessary to narrow the diameter of the PE pipe for the passage through the RIG by 30 tons.

Fig. 9: pushers

For safety reasons, a third pusher was installed midway. In fact, if during the installation of the new pipe, the forces on the winch would have exceeded a certain value, this pusher would have been put into operation.
In this case (which did not occur), the pusher n.3 would have integrated the action of the winch to pull the first 600 m.

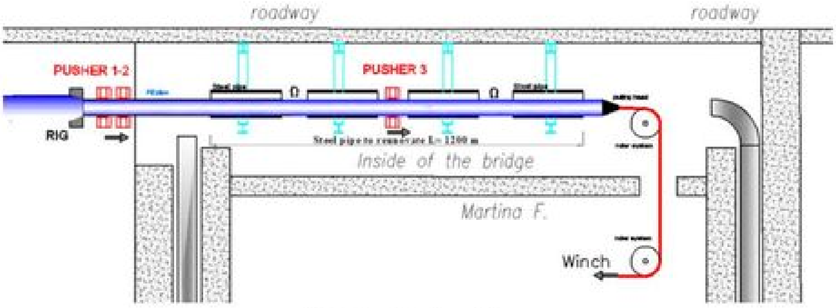

Fig.8: Installation scheme

The executive planning analyzed in detail the interaction of the intervention with the road infrastructure, with particular regard to the vertical and transversal actions generated by the pulling force due to the curvilinear plane-altimetric tracing of the existing pipes. Three types of forces were considered: longitudinal forces deriving from friction and the pulling force respectively and transversal forces from the plane-altimetric course of the bridge.
A 3D FEM model was used to carry out a simulation of the stress distribution and to identify the main stressed areas; as a result, the stressed areas are concentrated at the maximum curvature points and it pointed out that all the transversal stresses were directed towards the inside of the planimetric curves.

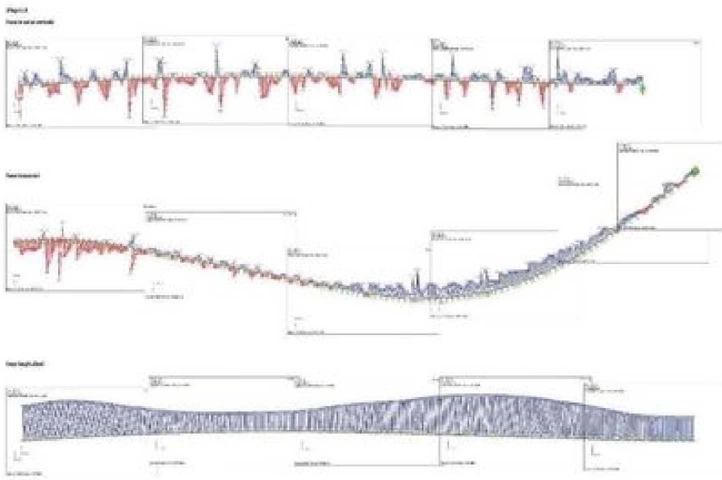

Fig.9: Simulation of the stresses

On the basis of the 3D model, three types of supports were calculated: one for highly stressed areas, one for medium forces and one for straight sections: Therefore, 85 shoring systems were built.
To avoid damages or concentrated stress, as for example the punching problem, due to the fact that the support system would have laid on very low wall-thickness sections of the existing pipe, saddle steel plates were installed in order to redistribute the acting forces. In contrast of the longitudinal force, created by the support on the hanging pipe, a steel carrier was placed every 50m. This was a very important step, because any damages or deformations would have blocked the new PE pipe. All of these elements were calculated and verified from a structural point of view, Figure 10.

Fig.10: prop system

One of the main aspects that affected the design of the layout of the site was the preparations, necessary to overcome the difference in level (about 9 m) between the road surface and the point of insertion of the PE pipe. Temporary support structures were built laterally outside the bridge to ensure that the pipeline reached the required insertion height, in correspondence with the existing opening in the bridge, as can be seen in Figure 11.

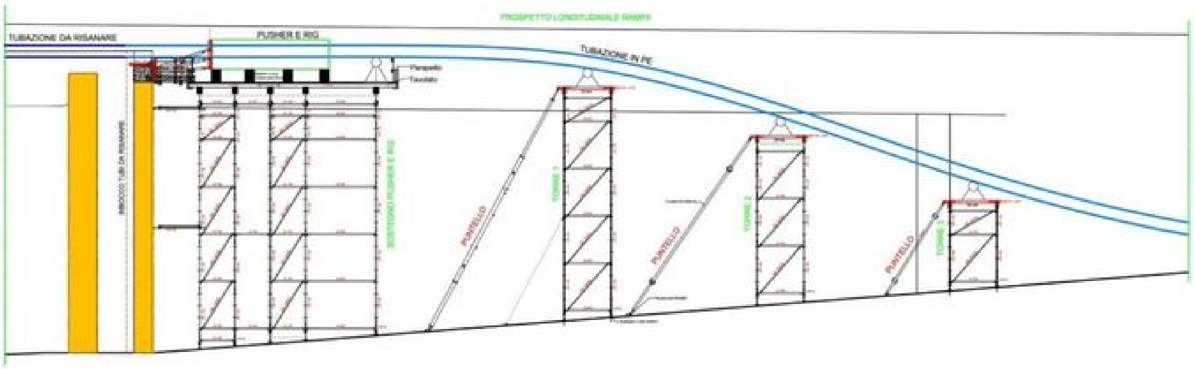

Fig 11: longitudinal profile of the insertion ramp

A head with an anchorage-hook was welded to the new PE pipe and resists the pulling action of the winch located at the opposite end.

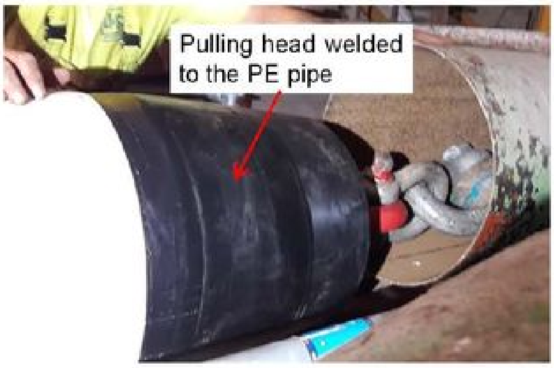

Fig.12: Pulling head

The winch was designed exclusively for this project and was equipped with a system of ropes and rollers to pull the pipes, Figure 13. Moreover, the design of a system of pulleys, arranged on a metal tower outside and inside the deck of the bridge, that allowed the cable to be diverted until it aligned with the axis of the pipe, was another fundamental aspect of the project.

Fig.13: The winch, with a pulling capacity of 100 t - system of ropes and rollers

Preparatory works to ensure the best possible installation result were undertaken. Those included the mechanical cleaning and the insertion of a test tube inside the pipeline to detect possible obstacles. The cleaning was managed opening different windows and it was alternated with the calibration of the pipe to verify that the internal section was constant and free of protrusions and deformations. The gauge consisted of a PE pipe Φ 480mm and at the exit it was visually inspected, to see if it had suffered damages.

The four new pipelines, after being welded piece by piece on site, brought to the entry point of the bridge and reduced in diameter through the rig, were pulled to the other end of the bridge, Figure 14. The company prepared 3 blocks composed by 20 PE pieces, each of them 20m long, so that 400m long sections at a time could have been welded.

Fig.14: PE pipe, pulling phase

The insertion and pulling phase of the approximately 1.200 m of new PE pipeline was carried out seamlessly, with an estimated time for the completion of the relining of an entire pipeline of about 20 hours. In the execution phase, the forecasts were substantially complied with. Below are some of the data collected during the relining process:

Pipe (N)	**Duration (hours)**	**Diameter RIG (mm)**	**Diameter PE Pipe (mm)**	**Diameter Reduction (%)**	**Temperature PE pipe (C°)**	**Max. Towing Force** (KN)	**Lengthening (%)**
2	20	440	500	12	17 - 24	500	4,0
3	14	450	500	10	10 - 20	500	2,1

Fig 15: data collected during the relining process

The rehabilitation was completed by the following activities:

- Stopping the pulling force and depletion of the retreat phase.
- Locking the PE pipe and restoring hydraulic connections.
- Reconstructing the structural continuity of the original pipeline.

After the insertion, the connections were restored, the replaced omega sections were substituted by special pieces that could deal with the thermal dilatation of the pipe.

Fig.16: Two of the four PE pipes, restored

The tender was delivered in May 2017 and completed after 9 months, in accordance with the contractual terms. The complexity of the project and its technical details involved national and international companies and the most competent experts in the close-fit sector. This significant intervention will guarantee the citizens an increasingly efficient service in line with the commitment of AQP in order to respond positively to the real expectations of the territory served.

In 2018 the project and its realization led to the winning of the "H_2O Award 2018", at the H_2O fair in Bologna, category pipes-manholes.

1st International NODIG Conference

Author Biographies

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Conference Chairman - Dr Dec Downey

After graduating from Bath University, Dec Downey joined the ARC Group in 1973 and spent 14 years involved with concrete pipe development and technology licensing, particularly in Japan and SE Asia. In 1987 he joined Insituform working until 2002 as Group Technical Manager and VP Asia Pacific. He was a partner at Jason Consultants Group in 2002-2010 before working as an independent consultant (Trenchless Opportunities Ltd) and has no plans to retire just yet!

He was a co-author of the NASTT CIPP Good Practices Guidelines and presently writes and delivers course materials for JBP Training. Dec became a member of ISTT in 1986 and UKSTT in 1993. He has been an ISTT Guarantor/Trustee since 1999, served as ISTT Chairman 2007-2010, and is currently Honorary President. His work in the concrete pipe business has been recognised by the Japan Microtunnelling Association with the Kurose Award, he is proud to hold a UKSTT Lifetime Achievement Award, an ISTT Fellowship and the ISTT Gold Medal.

Dr Olivier Thépot

Dr Olivier Thépot graduated from the Ecole Nationale des Ponts et Chaussées with a PhD in Geotechnical Engineering and he is also graduated in civil engineering and public works from the Ecole des ingénieurs de la Ville de Paris. He worked for 30 years at Eau de Paris (the Paris water public operator) in the engineering department as a scientific expert.
He has 30 years of experience in the diagnosis and the mechanical assessment of underground pipes and man entry sewers and in the design of structural rehabilitations.

He invented patented testing devices for the mechanical assessment of man entry sewers and a design method for non-circular linings.
He has been actively involved in elaborating the new French design Method for the rehabilitation of gravity pipes – the ASTEE 3R2014 Recommendations. He is also a member of the Blue Ribbon panel reviewers of the New ASCE Manual of Practice N°145 for the design of Close-fit Liners for Gravity pipes.
He is a member of the French Scientific and Technical Association for Water and Environment (ASTEE) and of the French Society for Trenchless Technology (FSTT).

Author Biographies

Nick Orman

Nick is a Chartered Civil Engineer and Environmental Manager, and Principal Consultant with WRc, specialising in the design, rehabilitation, management and operation of drain and sewer systems and in water industry standards and legislation. He has been involved in developing UK and European Standards for drain and sewer systems for over 30 years. His specialist areas include sewer inspection technologies, sewer deterioration mechanisms, sewer blockage mechanisms, standards development, product specifications and the practical understanding of relevant water industry legislation.

He is a major contributor to the forthcoming SRM Sewer renovation design guide.

Firmino Plácido Pires Barbosa

Is a graduated Civil Engineer, specialized in water and wastewater infrastructure as well as in waste disposal. After many years working as a consultant engineer, he changed to the contractor side and gained many years of experience as a site manager. This experience covered from the infrastructure, building, and housing sites and gave him the opportunity to improve his knowledge about general civil construction. n 1998 he moved into the technology of no-dig pipe rehabilitation for developing the international market for a German developed HDPE composed system. .

Following this the introduction to another no-dig pipe rehabilitation systems, e.g. ambient and hot curing felt liner systems and GRP segmental lining, was performed and knowledge improved in some job sites abroad.

Firmino joined RELINEEUROPE in January 2013 and is responsible for promotion the ALPHALINER, the RELINEEUROPE TQM (Total Quality Management) and, of course, the sales in the whole North, South and West European region.

His role is varied from the technical specification, product and material testing with water companies and their contractors.

Since 1998 involved in No-Dig pipe rehabilitation, he brings extensive experience of working on different CIPP technologies.

The foundation of RELINE APTEC – a new company in the RELINE UV Group – is since November 2016 the new additionally challenge. Firmino is in charge for the further development of the GRP UV-light curing hose liner for the renovation of pressure pipes and the business development for its application in different areas, e.g. sewer force mains, gas and potable water mains.

The skills of a civil engineer together with the experience in business development for trenchless pipe rehabilitation techniques grants Firmino the success within this new challenge.

Author Biographies

Dr Ricky Selle

Dr. Ricky Selle graduated in 2000 from the University of Leipzig as industrial engineer at the Institute for Solid Construction under Prof. Gert König. After working for KHP in Leipzig for one year, specializing in reinforced concrete and steel construction, he went to Ohio University in the USA for two years and worked on various research projects on the subject of concrete pavements. Back in his hometown, Ricky Selle worked as managing director in a planning office before founding his own architecture and engineering firm with a partner in 2006. An apartment building designed by his office in Leipzig was the tallest wooden house in the new German states (excluding Berlin) when it was completed in 2013, with four floors.

From 2011, Ricky Selle increasingly took on projects for Selle Consult in a parallel capacity. His focus was initially on static calculations of pipelines. In 2015, he became managing director alongside his father, and in 2016 he also became an owner of Selle Consult.

Since then, he has developed Selle Consult into one of the leading consulting companies for supporting structures of underground technical infrastructure made of plastics, helping to shape the relevant national and international regulations.

Since 2010, Ricky has lectured at the HTWK Leipzig within the master's program on the topic of the wood-concrete composite. He also was a lecturer at the University of Leipzig from 2014 to 2016 within the series of lectures on planning and operating municipal infrastructure.

In 2020, Ricky Selle was approved publicly appointed and sworn expert for the design of pipes and manholes including corresponding rehabilitation procedures.

Dr. rer.nat. Nils Füchtjohann

Dr Nils Füchtjohann trained at the University Dortmund from 1995 - 1996 before starting at the University Cologne in 1998 then Philipps-University Marburg before leaving in 2006.

Currently the Global Director for SAERTEX Nils has also been head of Research and Development and a developer for Schreiner Group.

Nils has a number of publications including:

Testing pressure CIPP under dynamic Loading; Trenchless International, winter 2018, Dr. Ricky Selle, Heiko Below and Dr. Nils Füchtjohann

Developing a Design Approach for CIPP under Pressure; Trenchless International, autumn 2017, Dr. Ricky Selle and Dr. Nils Füchtjohann

Author Biographies

Dr.-Ing. Mark Klameth

Mark Klameth is a civil engineer for geotechnical engineering with more than 17 years of professional experience. He studied civil engineering at the university of his hometown Hannover (Germany).
After graduating, he took up a position as a research assistant and doctoral student at the Institute for Geotechnical Engineering there. There he worked in cooperation with the IKT on his doctoral thesis on the MAC system, a non-destructive testing system for assessing the structural integrity of walk-in sewer collectors.
In addition to his work at the institute, he also worked as a freelancer for various engineering firms.

This included calculations for construction pit shoring, long-term settlement calculations of soft clay soils under container terminals and CPT monitoring for offshore wind energy plants. He also specialised in the calculation of special geotechnical problems using 3-dimensional FE programmes.
After completing his doctorate, he moved to IKT in Gelsenkirchen, where he has now been working for 7 years and is head of the structural analysis department.
In addition to calculating statics for CIPP liners and trench-laid pipes, his tasks also include supporting customers with special geotechnical problems, conducting product tests and seminars in the field of geotechnics and occasionally giving lectures at the universities in Hanover and Bochum in the field of sewer and pipeline construction.

Dr. Steve Brogden

Dr. Steve Brogden is a Polymer Engineer with 40 years of experience in materials technology, having specialised in plastic pipes for over 30 years.
He was involved in the development of polymer pipes and lining systems for the water industry in the mid-1990s, contributing to Water Industry Specifications. His PhD on the fatigue loading of PVC and PE pipes formed the basis of a UK Water Industry IGN.

Between 2000 and 2008 he worked as a plastic pipe specialist in Sweden and Scotland. Between 2009 and 2018 Steve was the Technical Authority for Swagelining, being responsible for all Die Draw liner design calculations globally. This included design for a wide range of applications including high pressure subsea water injection pipelines and large diameter utility water pipes.
Steve has been Managing Director of Die Draw Ltd from its inception in 2018 to date, with the mission of re-introducing Die Draw technology to the water and gas utility industries globally. Temporary works solutions for implementation of Die Draw in major cities where access is restricted is key to the ongoing successful deployment of the technology, especially for the UK water industry.